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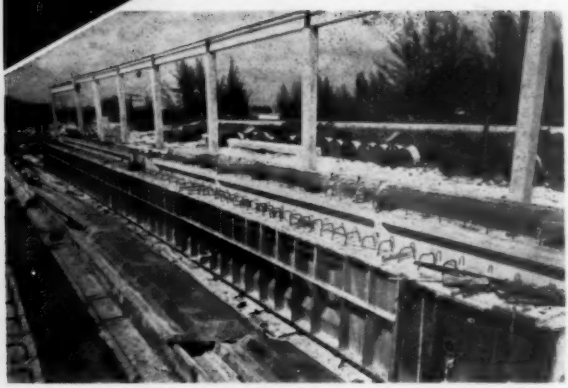
10 1/2 MILES of JOISTS

produced from one set of WATCO forms



The casting yard of R. H. Wright & Son, showing the 430-ft. long WATCO steel form used to produce the more than 75,000 lin. ft. of prestressed concrete I-joists since installation in 1955.

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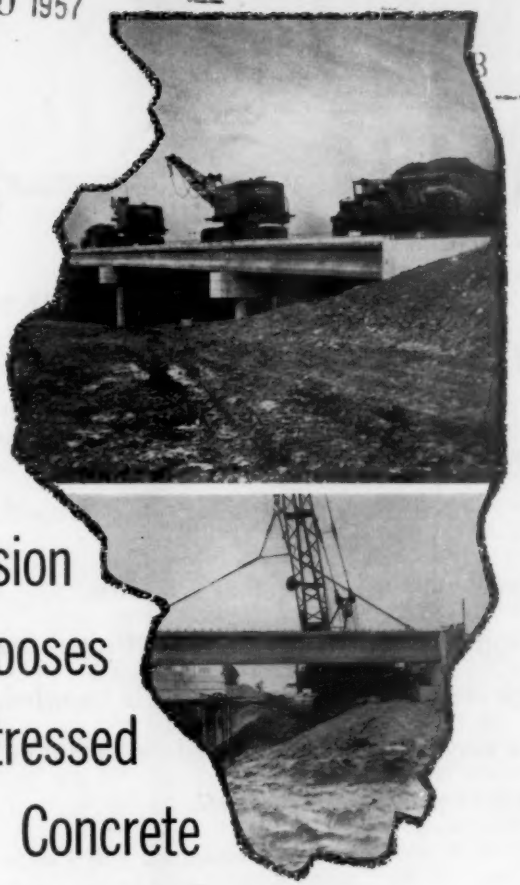

Overload test of Beverly Road Bridge. Four spans, continuous under live loads, center to center of piers measured along center line of girders are: 42'-10"; 70'-5"; 70'-5"; 42'-10". Skew is 20°. Precast prestressed I-section girders—without intermediate diaphragms—are 48" deep and placed 7'-6" center to center. Precast, pretensioned 2½" deck slabs were placed on the girders with a 5" slab poured on top. Intermediate piers are 36" diameter prestressed hollow piles with 4" walls.

Girders and 2½" slabs for the test bridge made by Prestressed Concrete Structures of Frankfort, Illinois, using Roebling 7-wire stress-relieved strands.

Placing 42'-10" span girder on Beverly Road Bridge.

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The 5-day Conference will bring together scientists,
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science and industry of prestressed concrete.

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Registration fee: $25.00 (U. S. Currency).

For reservation forms write to:
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JOURNAL

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From the editor . . .

The primary functions of the PCI JOURNAL are: the exchange of ideas among the members of the Prestressed Concrete Institute; to further the knowledge, application, production and use of prestressed concrete. To achieve this goal, members from every phase of the industry should make frequent contributions, technical and non-technical, to the JOURNAL.

Items that would be of interest to our many readers include new and unusual applications, results of theoretical and experimental research, production problems, and transportation and construction problems involving extraordinary features. Photographs or drawings of structures having unusual features would also be welcome contributions.

The success of this publication depends on the interest and active participation of the members of the Institute and our readers. Let us hear from you often; we welcome any news of the prestressed concrete industry and your organizations.

The **JOURNAL** is published quarterly by the **PRESTRESSED CONCRETE INSTITUTE, INC.**, 425 N. E. 5th Street, Boca Raton, Florida. **TECHNICAL EDITOR:** Dr. A. M. Ozell, University of Florida, Gainesville, Florida. **ADVERTISING, SUBSCRIPTION** and **PUBLICATION OFFICE:** 3132 N. E. 9th Street, Fort Lauderdale, Florida. **DISTRIBUTION:** Available without cost to all members of the Prestressed Concrete Institute. Additional copies offered to members at $1.00 per copy. **SUBSCRIPTION PRICES:** Single copies: $2.00. Subscription rate: $6.00 per year for four issues. For subscriptions outside the continental limits of the United States, add 20% to cover handling and mailing.

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SOME STUDIES ON THE DEFLECTION

of Prestressed Concrete Beams

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and

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INTRODUCTION

There are several factors which influence the modulus of elasticity of concrete, notably the strength of concrete, the properties and size of the aggregates, type of cement, the curing method, and the definition of modulus of elasticity itself, whether tangent, initial, or secant modulus. The modulus may vary also with the speed of load application and with the type of specimen, whether a beam or a cylinder. With all these variables it is almost impossible to predict accurately the value of the modulus for a given concrete.

For low strength concrete, as employed in conventional reinforced concrete construction, it was found convenient to use empirical formulas to determine the value of the modulus of elasticity as a function of the ultimate strength. The results were reliable for low strength concrete but for high strength concrete, they were not at all in agreement with the actual behavior.

In general, test data on prestressed concrete beams indicate that the values of the modulus of elasticity determined from measured deflections are higher than those determined from measured strains. This observation gives rise to many questions. What, if any, is the relationship between the "deflection modulus" and the "strain modulus"? Can these moduli be related to the cylinder strength of the concrete? What is the relationship between the ultimate strength of concrete in compression and in flexure?

The purpose of this study was to define more accurately the behavior of prestressed concrete in the so-called elastic range, or before cracking. A series of beams of various lengths, cross sections, and prestress were tested and strain distribution and load-deflection measurements were taken. Test data on prestressed concrete published by other investigators were also collected. From this data a comparison was made with the theoretical approach, and the various assumptions checked for accuracy.

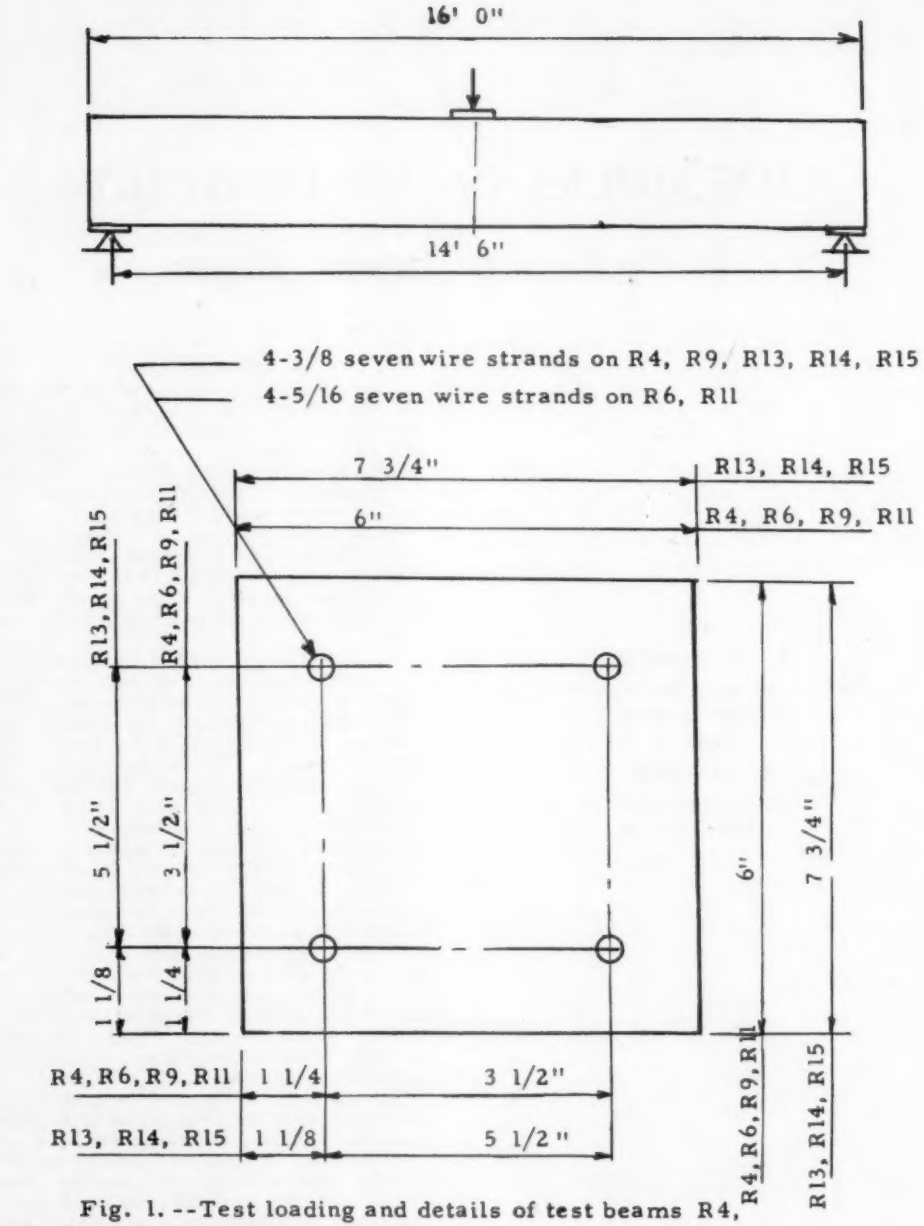
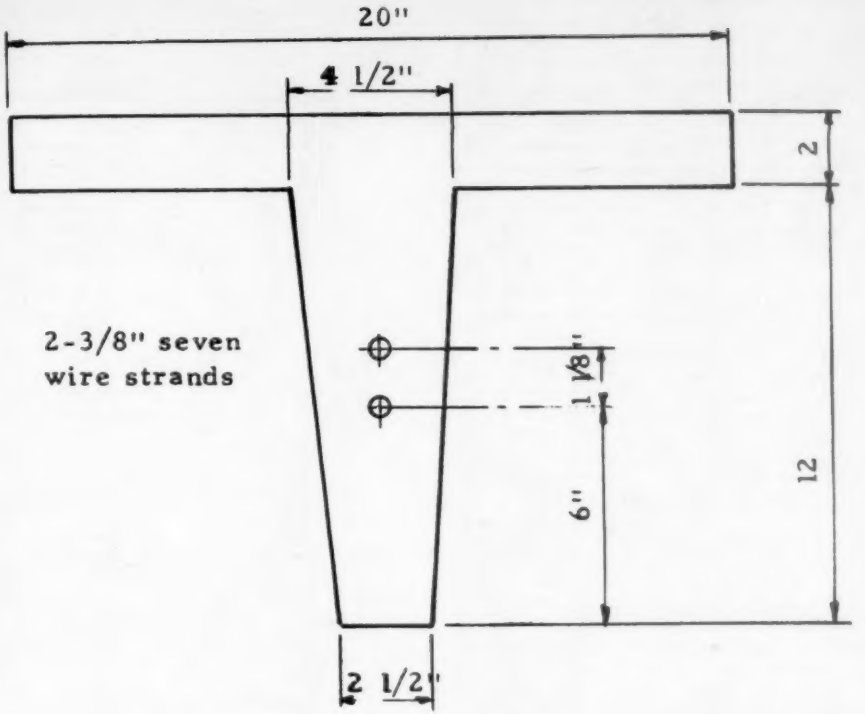
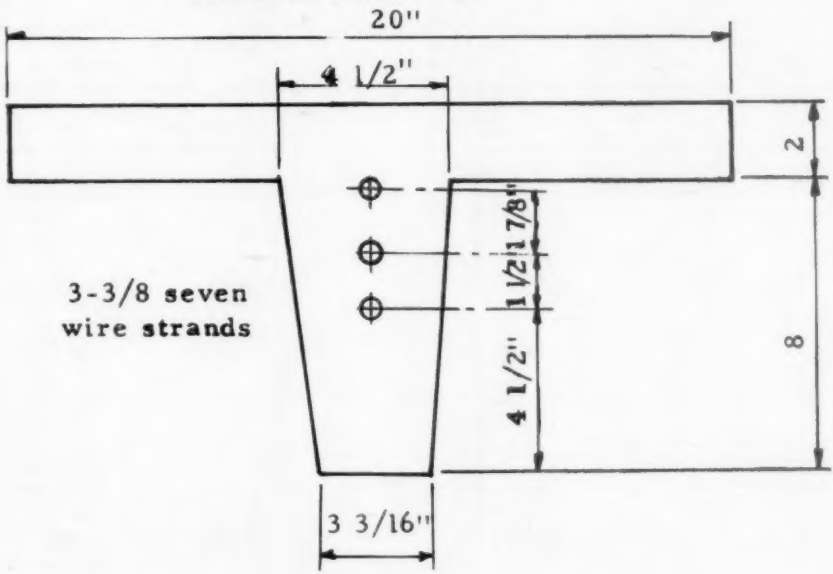


Fig. 1. --Test loading and details of test beams R4, R6, R9, R11, R13, R14, and R15.



Beam T-1 and T-2



Beam T-3 and T-4

Fig. 2.--Details of test beams T-1, T-2, T-3, and T-4

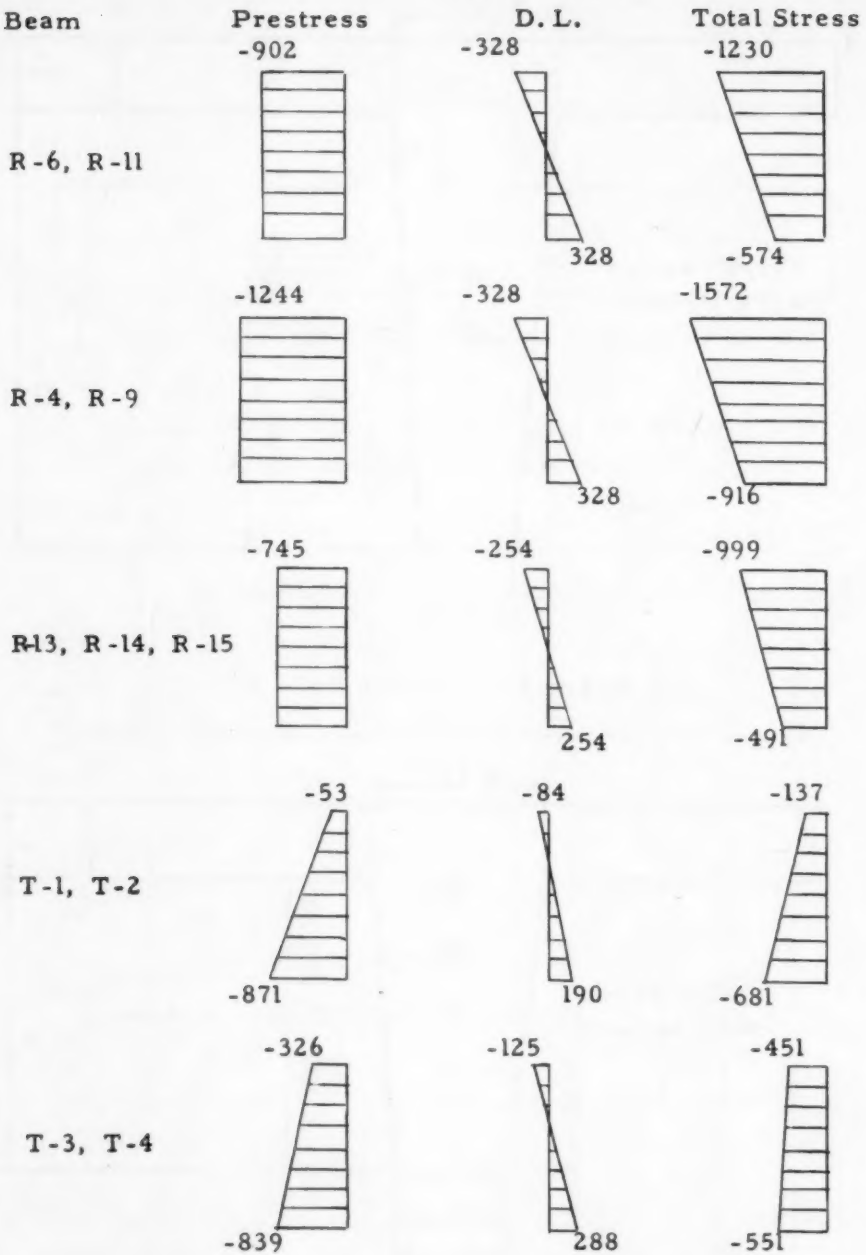


Fig. 3.--Calculated stress distribution before loading - R and T Series (20% stress loss)

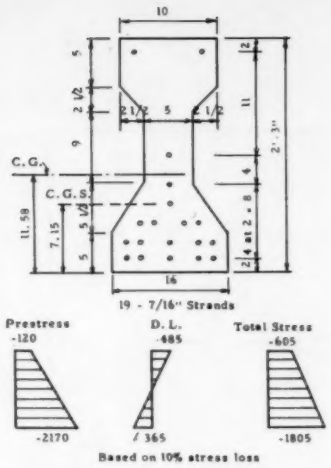


Fig. 4. --Cross section of beams of I Series and calculated stress distribution before loading

An effort was made to develop a theory which will accurately predict deflections and strains in prestressed concrete beams in flexure. Comparisons were made between the developed theory, the straight line or the elastic theory, and the actual measured values.

DESCRIPTION OF SPECIMENS

Some of the beam specimens (R-Series) had square cross sections, as shown in Figure 1, with an overall length of 16 ft. The cylinder tests indicated an average ultimate strength of 8000 psi and an average initial modulus of elasticity of 5.15×10^6 psi. In all cases, at least three cylinders were tested.

Other beam specimens (T-Series) were tee-sections with cross sections as shown in Figure 2 and an overall length of 15 ft. The test set up was the same as shown in Figure 1. The cylinder tests gave the following results:

Average ultimate strength, f'c 7580 psi
Average initial modulus of elasticity 4.75×10^6 psi

The calculated stress distribution for R and T Series before loading is shown in Figure 3.

Also included in this study are test results of two bridge I-beams 27 in. deep, having 40-ft. span and pretensioned with nineteen 7/16-in., seven-wire strands. The cross section of these beams, I-21S and I-31S, and the stress distribution before loading are given in Figure 4. The cylinder tests gave the following results:

Average ultimate strength, f'c 5200 psi
Average initial modulus of elasticity 4.3×10^6 psi

In calculating the stresses due to pretensioning, 20 per cent stress loss was assumed for R and T Series since the beams were about 100 days old when tested, and 10 per cent stress loss was used for I Series as I-21S was two days and I-31S was three days old when tested.

DEVELOPMENT OF THEORY

Basic Assumptions

In flexural analysis of prestressed concrete below the cracking load, the straight-line theory has been accepted as a reliable method of determining stress-strain relationships. The main advantages of this theory are its simplicity and adaptability to design. For conventional reinforced concrete this idealized stress-strain curve is appropriate because the allowable stresses are limited to such low values that for all practical purposes the stress-strain curve can be assumed to be a straight line. This assumption is not necessarily valid in prestressed concrete, however, since in some conditions of prestressing it is possible to have very high compressive stresses in the top fiber of a beam and still little or no tensile stress in the bottom fiber (Figure 5). With this condition the stress level is no longer within the elastic portion of the curve (Figure 6).

The theory developed here is based on the assumption that the stress-strain curve of

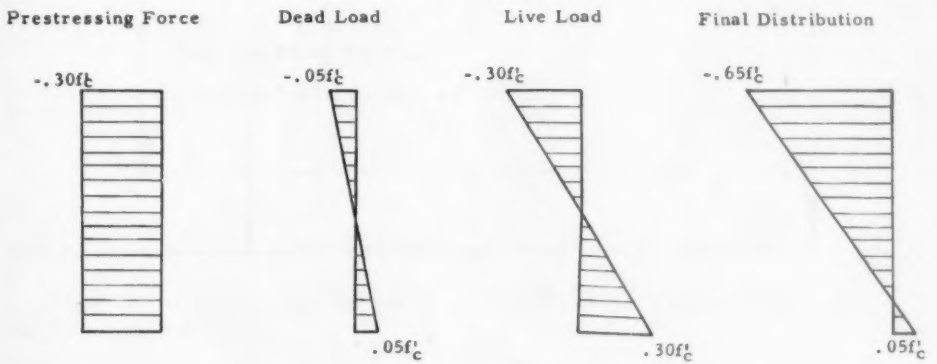


Fig. 5.--Typical stress distribution

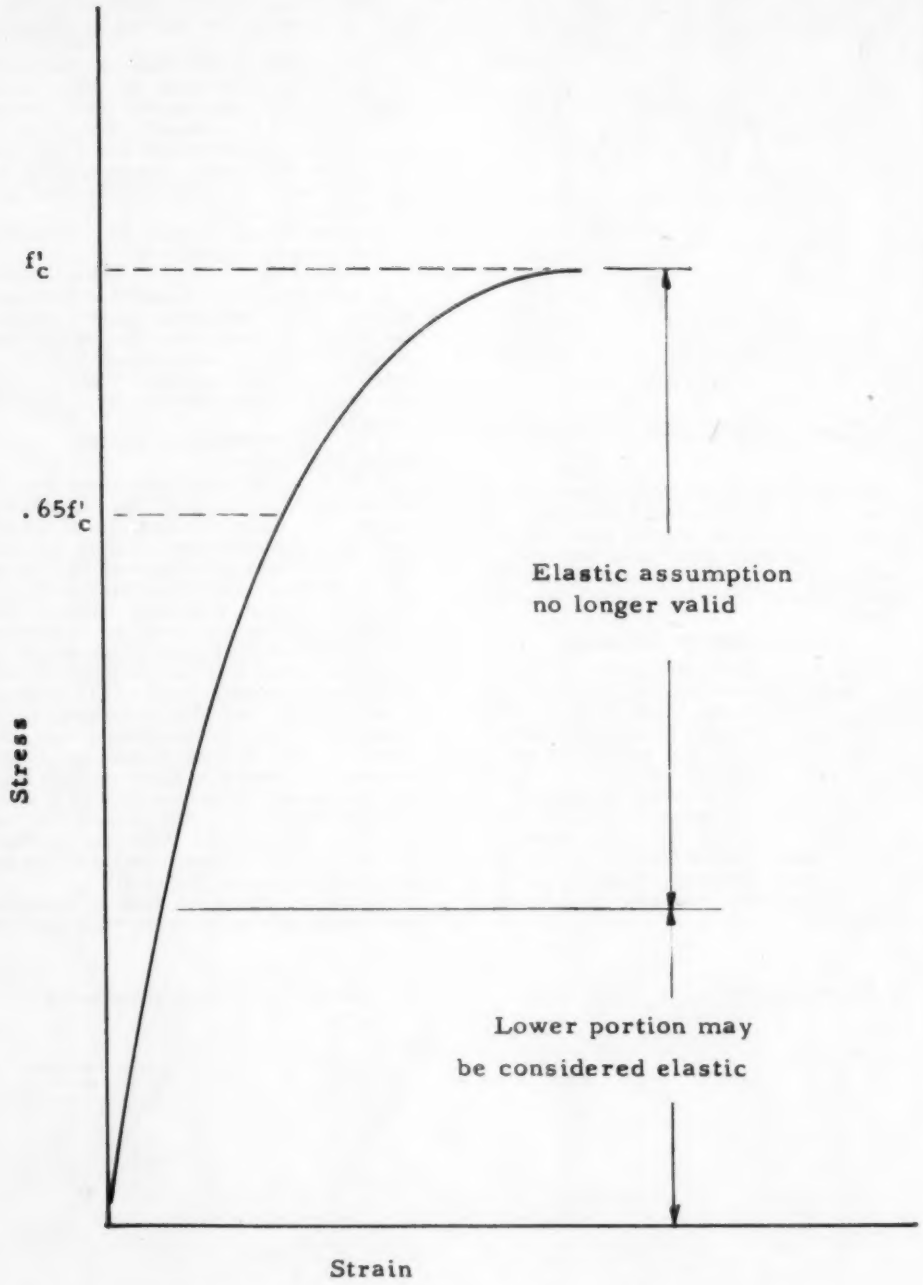


Fig. 6.--Typical stress vs. strain curve

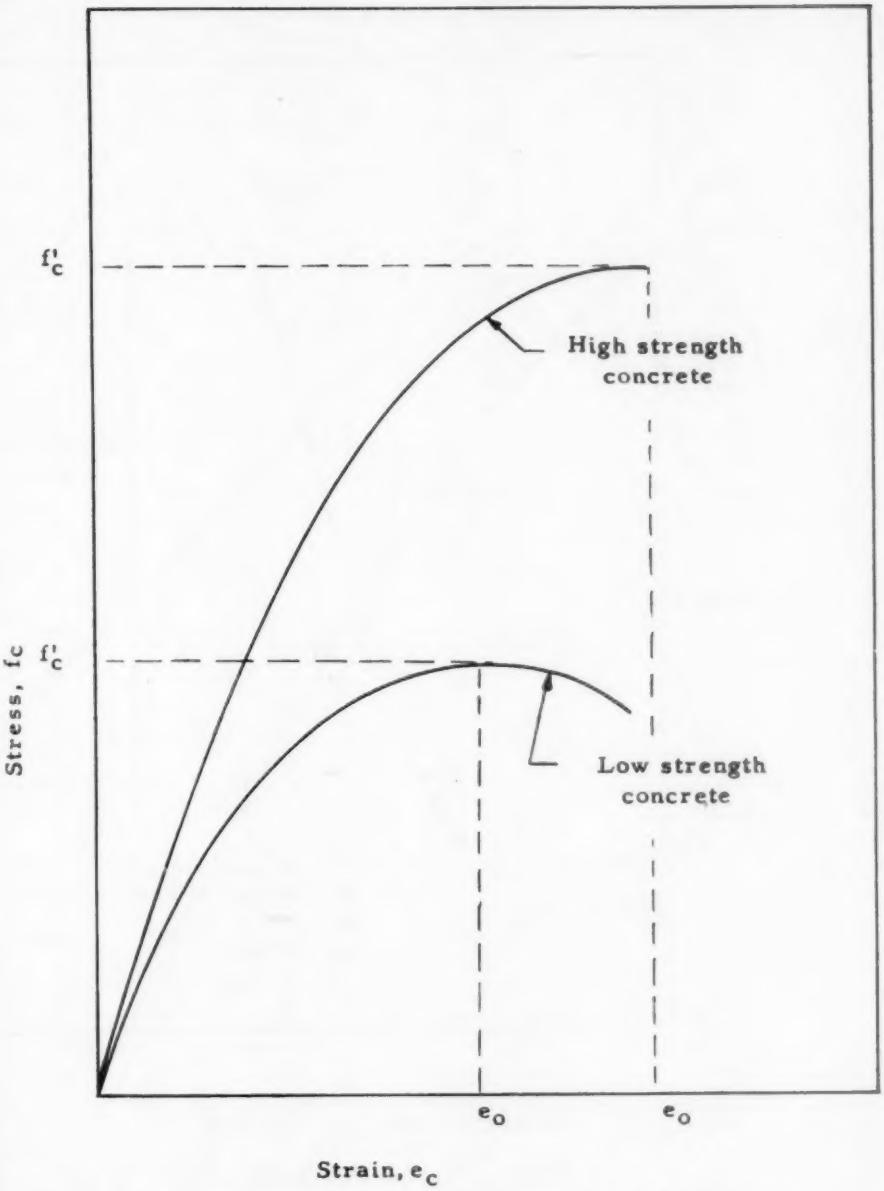


Fig. 7.--Comparison of stress vs. strain curve for high and low strength concrete

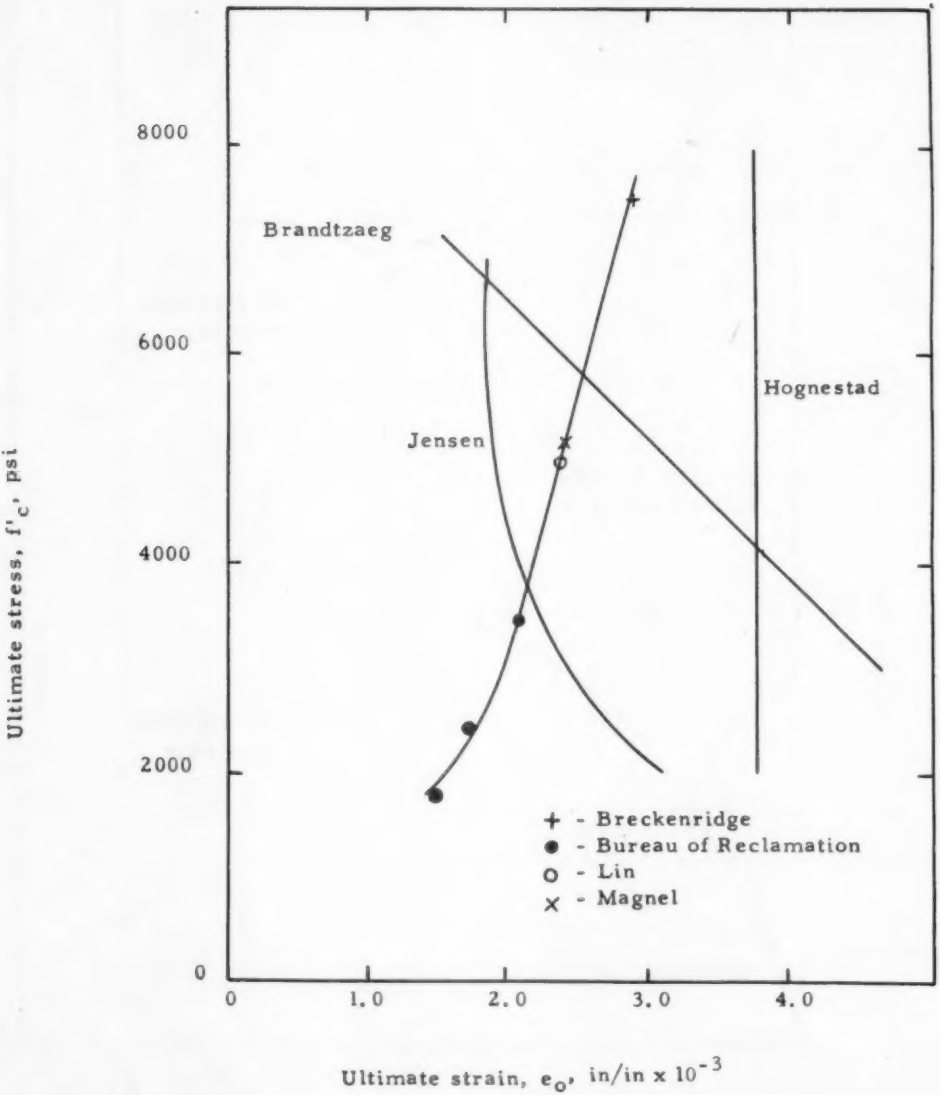


Fig. 8. --Relationship between ultimate stress and ultimate strain in compression of concrete.

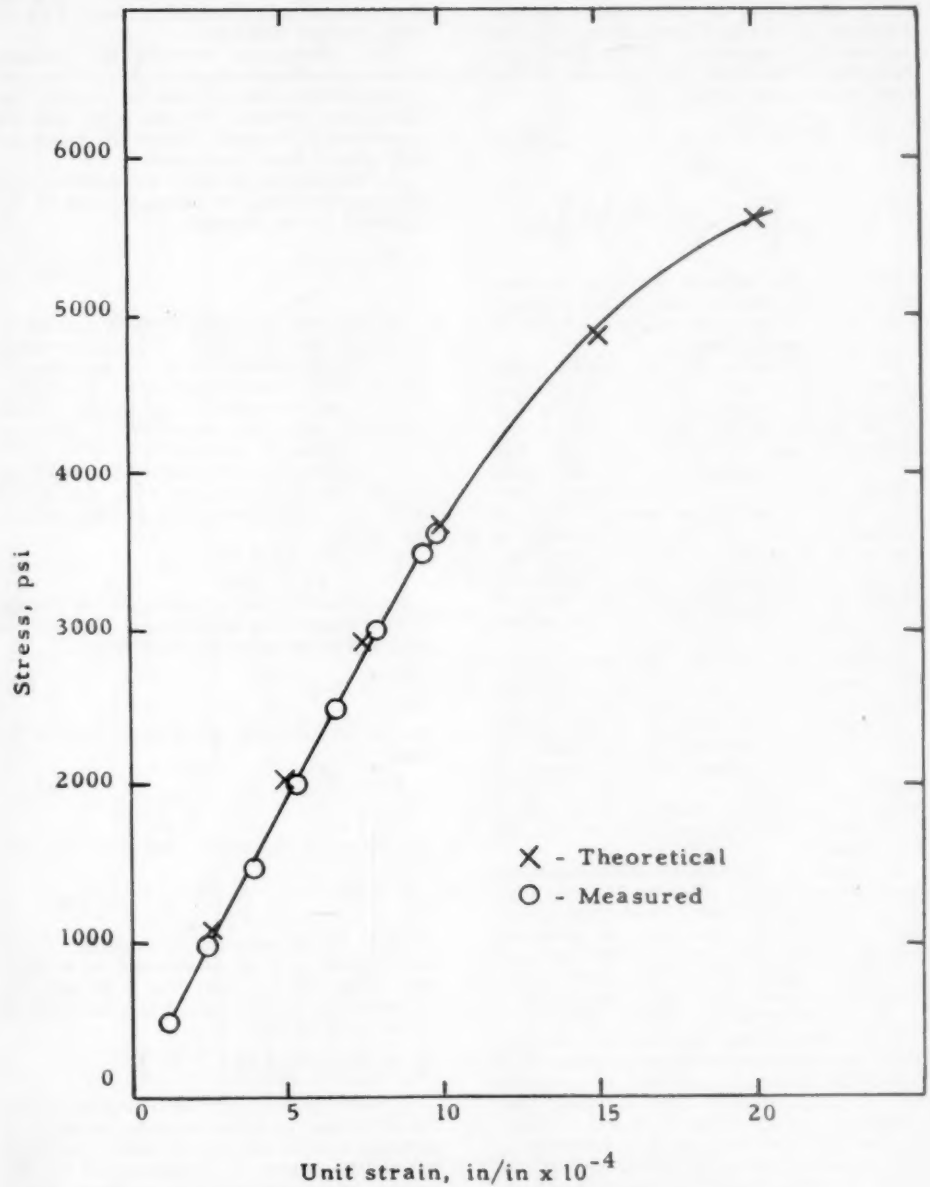


Fig. 9.--Comparison of measured and theoretical stress vs. strain curve, f'_c = 5840 psi.

concrete, both in compression and in flexure, is a second degree parabola. This is by no means a new concept since other investigators (1, 2) have assumed parabolic distribution of stresses in flexure.

Since the ultimate strength of a concrete mix is determined by compression tests of cylinders, an idealized stress-strain curve of concrete in compression will be developed. Assuming a parabola, the equation of the stress-strain curve would be:

$$fc = \frac{2f'c}{e_o} e - \frac{f'c\ e^2}{e_o^2} \qquad (1)$$

or

$$e = e_o \left[1 - \left(1 - \frac{fc}{f'c} \right)^{1/2} \right] \qquad (1a)$$

where f'c is the ultimate stress in compression, eo is the strain at the ultimate stress, fc is the stress at any point on the curve and e is the corresponding strain.

In order to define the stress-strain curve for any given concrete strength, a relationship must be determined between f'c and eo. In this study the ultimate strain is defined as the strain at the ultimate stress. For high strength concrete the ultimate strain actually will be the strain at failure. However, this is not so for low strength concrete (Figure 7). Concrete strengths included in this study range from 5000 psi to 8000 psi and will be designated as high strength, although the actual transition point between a high and a low strength conerete, as defined by Figure 7, is not known.

Available data on the ultimate compression strains in concrete is rather conflicting. Some of these are reproduced in Fig. 8 for a comparison. The variation in the results reported could be attributed to the different concretes used, instrumentation, speed of loading and techniques in testing. For this study, a straight line relationship for values of f'c above 3000 psi as shown in Fig. 10 was assumed.

In Figure 9, the theoretical stress-strain curve determined by equation (1) and Figure 8 is compared with a measured stress-strain curve. In this case the ultimate strength of the concrete was 5840 psi and the ultimate strain was 25.5 x 10^{-4} in./in. Substituting these values in equation (1):

$$fc = 4.58 \times 10^6\ e - 898 \times 10^6\ e^2$$

It is apparent from Figure 9 that the parabola is a reasonable approximation of a stress-strain curve for concrete in compression. It has generally been accepted that stress-strain relationships obtained in compression tests may be applied to flexure with good accuracy, hence, a second degree parabola is assumed here for the stress-strain curve in flexure. The maximum stress in flexural compression, f"c, is assumed to equal 90 per cent of f'c. The value of 90 per cent was taken from test data by Magnel (3) who reported a maximum compressive strain of 12.8 x 10^{-4} in./in. and a maximum tensile strain (before cracking) of 0.375 x 10^{-4} in./in. These strains were measured on a prestressed concrete beam with a cylinder strength of approximately 5200 psi. It was found that in order to be consistent with the measured strains and deflections, the maximum stress in flexural compression had to be approximately 0.9 f'c. Hence, for this study it is assumed that f"c is 90 per cent of f'c for high strength concrete.

The relationship between the ultimate stress and ultimate strain in flexure was determined from the test data to give the best correlation between the measured and the calculated deflections. Figure 10 shows the relationship thus determined.

By assuming a parabolic stress-strain relationship, the initial modulus, Ef, can be determined by the formula:

$$Ef = \frac{2\ f''c}{e_o} \qquad (2)$$

At this point it might simplify matters to point out that the slope of the stress-strain curve will be referred to as the modulus, E, and since the curve is parabolic, the modulus will not be constant and should not be confused with the modulus of elasticity which is generally denoted by E.

Substituting the relationship obtained from Figure 10 into equation (2), the initial modulus can be determined as only a function of f"c:

$$Ef = \frac{2f''c\ (5 \times 10^6)}{f'c + 7000} \qquad (2a)$$

By differentiating equation (1) the slope of the compression stress-strain curve or the modulus at any point can be defined:

$$\frac{dfc}{de} = E = \frac{2f'c}{e_o} - \frac{2f'c}{e_o^2} e \qquad (3)$$

or for the flexural stress-strain curve, we have:

$$E = \frac{2f''c}{e_o} - \frac{2f''c}{e_o^2} e \qquad (3a)$$

and by using equations (1a) and (2), we get:

$$E = Ef \left(1 - \frac{fc}{f''c} \right)^{1/2} \qquad (3b)$$

Finally, by substituting in equation (2a), the modulus can be determined as a function of fc and f"c only; that is to say, the modulus can be determined as a function of the stress level only:

$$E = \frac{10 \times 10^6\ f'c}{f''c + 7000} \left[1 - \frac{fc}{f'c} \right]^{1/2} \qquad (3c)$$

In order to determine an effective modulus to be used in deflection calculations, it is assumed that the effective E will be an average of the values of E determined by the stresses at the top and bottom fibers. Since the two values will be the extreme values, it seems that an average of the two extremes would be the likely value.

It should be noted, however, that due to loading conditions, the portion of the beam subjected to extreme fiber stress could be relatively small. In such cases an arithmetic mean stress would be a more proper value to

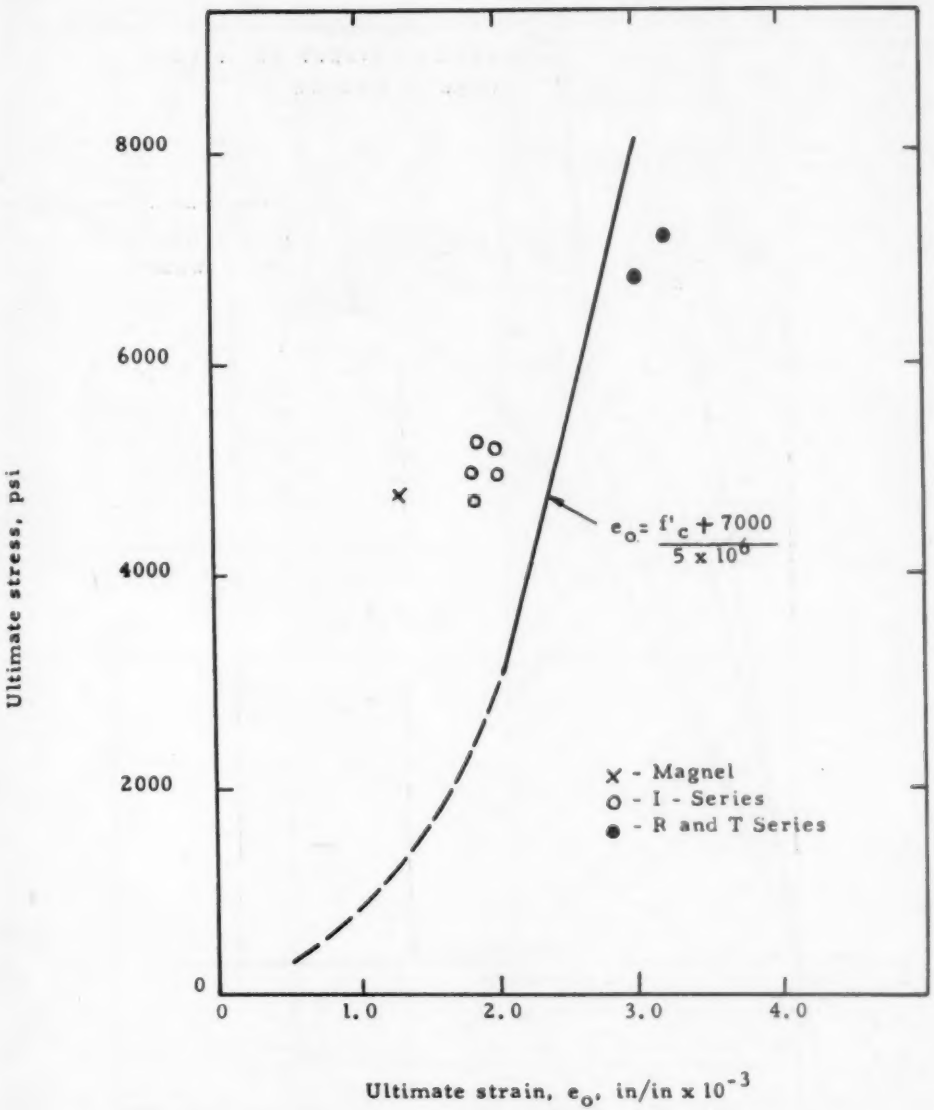


Fig. 10. --Relationship between ultimate stress and ultimate strain in flexure

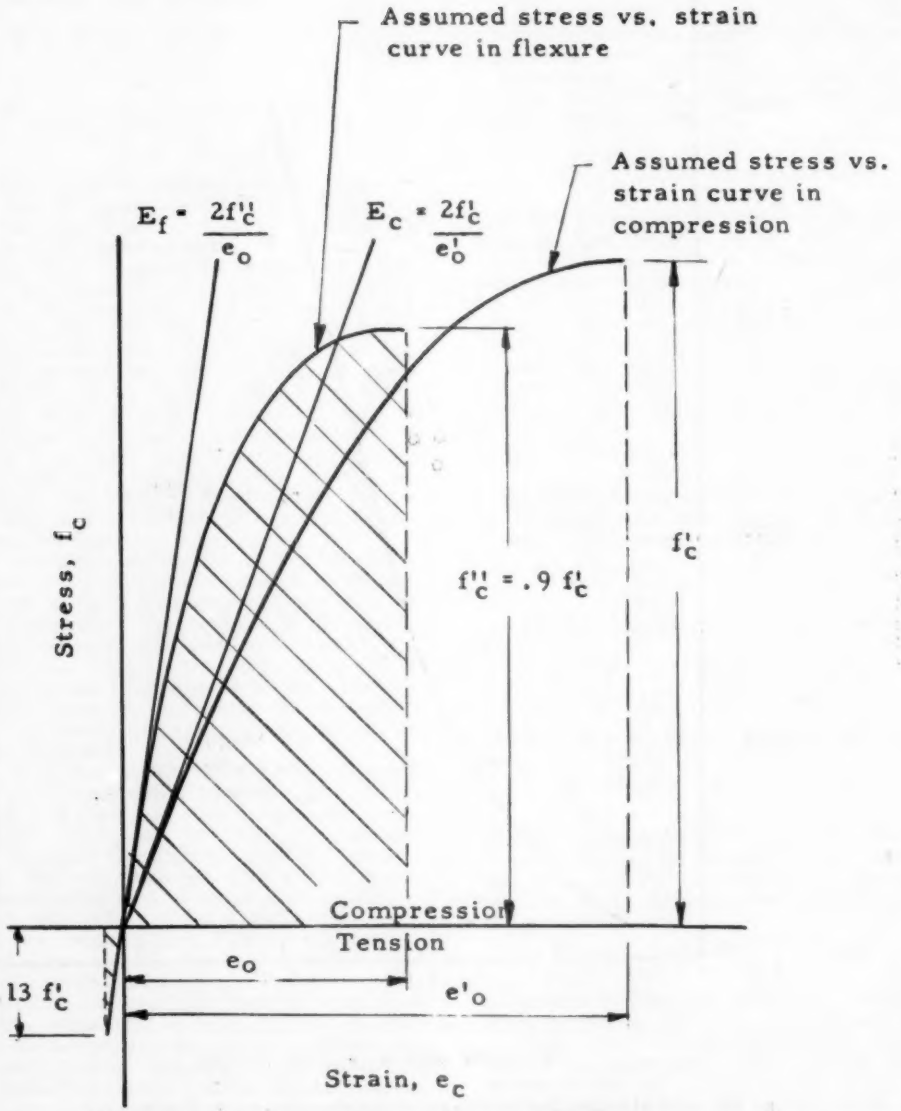


Fig. 11.--Theoretical stress-strain relationship of concrete

use; however, such a refinement would be cumbersome and not justified since many unknowns in the concrete properties of any given beam could offset such an improvement.

In many cases the bottom fibers of a beam will be in tension and it is therefore necessary to extend the flexural stress-strain curve. The tension portion of the curve is assumed to be a continuation of the flexural compression curve.

The direct tensile strength of concrete is a highly variable item generally ranging from .06 f'c to .10 f'c (4) and may be zero if cracks have developed as a result of shrinkage. The modulus of rupture in concrete is known to be higher than its direct tensile strength, varying from about .10 f'c to .15 f'c (5). For this study an average value of .13 f'c was taken for the maximum flexural tensile strength and, as soon as calculated tensile stress exceeds this limit, it is assumed that the section has cracked and the theory no longer applies. The value of E on the tension portion of the curve can still be determined from equation (3c) but tensile stresses must be taken as negative, thus changing the sign under the radical. Hence, the values of E for tensile stresses are larger than the initial modulus. This seems logical since test results have shown that the modulus is larger in decompression than in compression.

The final assumed stress-strain relationship is shown in Figure 11.

Applications

The main purpose of developing this theory was to provide a method of determining a value of E which could be used with reasonable accuracy in deflection calculations of prestressed concrete beams. In developing the expression for E, a theoretical value for E was derived first so that it would be possible to compare theoretical and measured strains as well as deflections.

Although the expression for E contains only two variables, fc and f'c or f''c, it is still rather cumbersome to determine. Therefore, it might be more convenient to draw a series of curves as shown in Figure 12, where the value of E can be selected for any stress level and for several strengths of concrete. Where the curve crosses the abscissa is the value of the initial modulus for the given f'c. The positive ordinates denote compressive stress levels, the negative denote tensile stress levels.

Example

As an example of how the theory is applied, one of the test specimens will be analyzed by the straight line theory and then by the use of the effective modulus.

Taking test specimen T-1, the stress in the top fiber due to prestress and dead load is 137 psi compression and that in the bottom fiber is 681 psi compression. With a superimposed load of 1860 lbs. the stress in the top fiber is 315 psi compression and that in the bottom fiber is 117 psi compression. An average of six cylinders gave f'c = 7580 psi and an initial modulus of elasticity of 4.75×10^6 psi.

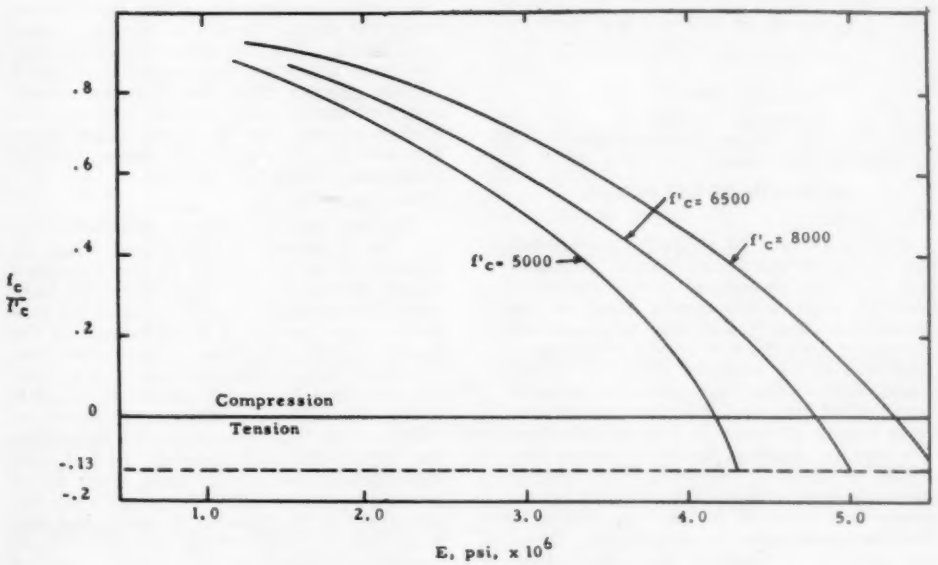


Fig. 12. $\frac{f_c}{f'_c}$ vs. E

Straight Line Theory

$$\text{deflection} = \frac{1860 \times (174)^3}{48 \times 4.75 \times 10^6 \times 1374}$$

$= .0313$ in.

Strains due to live load:

$$\text{Top fiber } e = \frac{315}{4.75 \times 10^6} - \frac{137}{4.75 \times 10^6} = 3.74 \times 10^{-5}$$

$$\text{Bottom fiber } e = \frac{681}{4.75 \times 10^6} - \frac{117}{4.75 \times 10^6} = 11.89 \times 10^{-5}$$

Effective E Method

$$\text{Top fiber} = \frac{fc}{f'c} = \frac{315}{7580} = .0416$$

from Figure 12, $E = 4.26$

$$\text{Bottom fiber } \frac{fc}{f'c} = \frac{117}{7580} = .0155$$

$E = 4.35$

$$\text{effective } E = \frac{4.26 + 4.35}{2} = 4.305 \times 10^6$$

$$\text{deflection} = \frac{1860 \times (174)^3}{48 \times 4.305 \times 10^6 \times 1374}$$

$= .0344$ in.

Strains due to live load:

$$\text{from Figure 10, } eo = \frac{0.9 \times 7580 + 7000}{5 \times 10^6}$$

$= 2.77 \times 10^{-3}$

Top fiber using equation (1a):

e_1 for fc of 137 $= .0316 \times 10^{-3}$

e_2 for fc of 315 $= .0727 \times 10^{-3}$

$e = e_2 - e_1 = 4.11 \times 10^{-5}$

Bottom fiber:

e_1 for fc of 681 $= 16.1 \times 10^{-5}$

e_2 for fc of 117 $= 2.5 \times 10^{-5}$

$e = e_1 - e_2 = 13.6 \times 10^{-5}$

Test Results

Deflection = .0342 in.

Top fiber strain, $e = 2 \times 10^{-5}$

Bottom fiber strain, $e = 14 \times 10^{-5}$

DISCUSSION OF TEST RESULTS

Deflections

Figures 13 and 14 show the load-deflection curves for the square beams. It can be seen that the theoretical curves compared favorably with the test results. However, the straight-line theory also was in reasonable agreement with the test results. This agreement was to be expected since the square beams had a very high ultimate strength, $f'c = 8000$ psi, and had no eccentricity. From Figure 10 it can be seen that just before rupture, strain in flexure increases very rapidly as the stress increases. Therefore, an extremely high strength concrete will have a stress-strain curve in flexure very similar to the stress-strain curve in compression. Hence the modulus of elasticity determined from cylinder tests will be very close to the value of the initial modulus in flexure, and the deflection determined by the elastic theory will approximate the value determined by the parabolic theory. This is especially true at low stress levels, which was the case for the square sections. Since there was no eccentricity of the prestress force, the compressive stresses were relatively low at the time the section cracked.

Figures 15 and 16 are for the tee-sections and again because of the high ultimate strength, $f'c = 7580$ psi, and the low stress level, both the elastic and the parabolic theories are in reasonable agreement with the measured deflections. In Figure 15, the fact that beam T-2 was cracked before testing explains the large deviations in the upper portion of the curve.

Figures 17 and 18 give the load-deflection relationship for the I-sections. These sections had ultimate strengths of 5200 psi and relatively high stress levels at time of cracking. It is believed that the parabolic theory gives much better agreement with the measured deflections.

It is obvious that the parabolic theory cannot be justified for extremely high strength concretes and low stress levels since the elastic theory gives reasonable results and is much simpler. For other than extremely high strength concrete and low stress levels, however, the effective E determined by the parabolic theory appears to be more accurate.

Strains

The theoretical and measured strains for I-Series are presented in Table 1. The strains determined by the elastic theory are, in general, slightly higher than the measured strains, while the strains determined by the developed theory tend to be slightly lower. In Figure 19 a comparison is made of the theoretical and measured stress-strain curves, using the average values of the four beams in Table 1. This comparison indicates that the theoretical stress-strain curve in flexure is slightly steeper than the true stress-strain curve in flexure and that the true curve is slightly steeper than the elastic curve where the modulus of elasticity is determined by compression tests of cylinders.

Developed Theory

The test results indicated that deflections can be accurately predicted by using an effective E as determined by the developed theory. It was also determined that the theoretical stress-strain relationship does not present a true picture of the behavior of the beams. If the true stress-strain curve were less steep than the assumed curve, it is obvious that the effective E would be smaller; hence the calculated deflections would no longer agree with the measured deflections and would yield higher values. In that case, there would have to be some other factor tending to reduce the deflection. In the development of the theory, the assumption was made that the only variable affecting the deflection of a flexural member before cracking was the modulus and that the modulus could be determined solely from the stress level of the beam at any instant. It is quite possible that this assumption was erroneous.

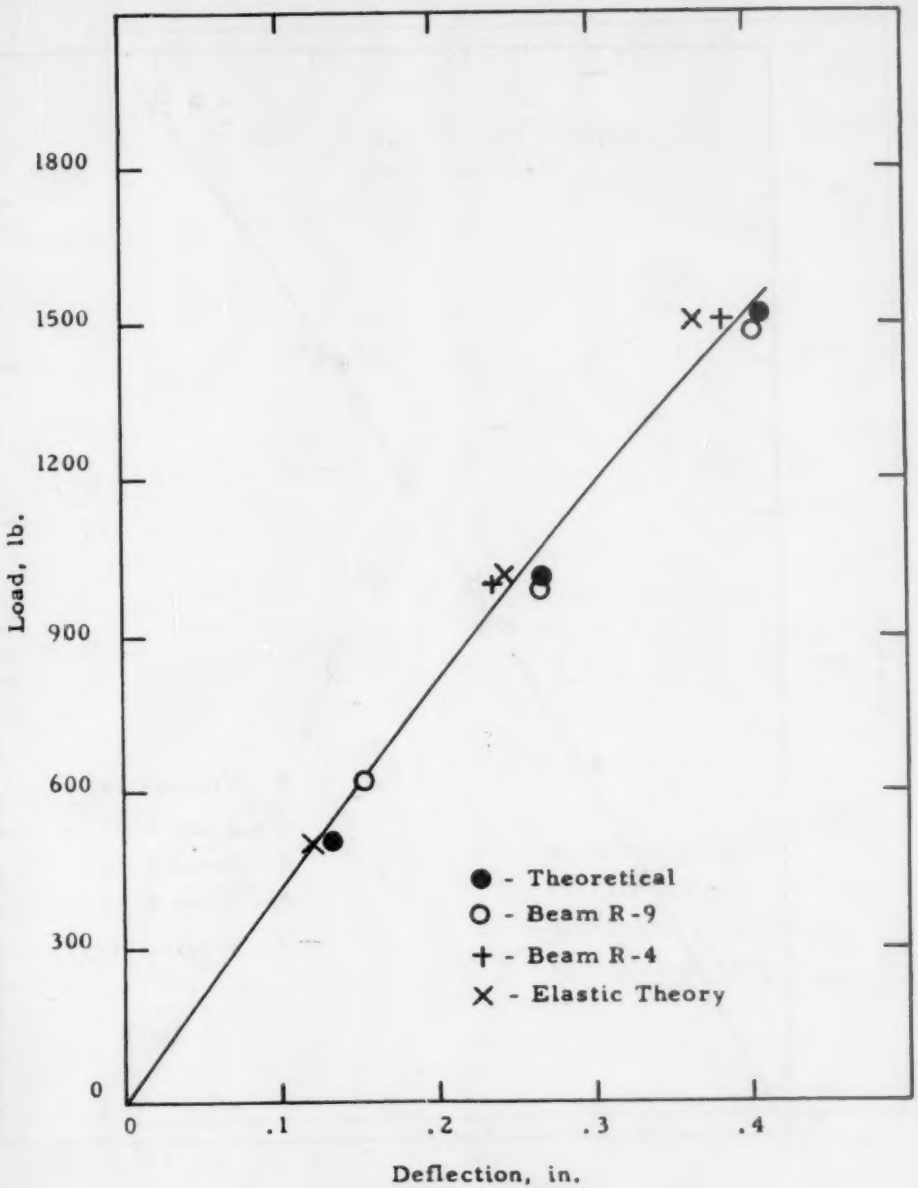


Fig. 13.--Comparison of theoretical and measured load-deflection curve, Beam R-4 and R-9

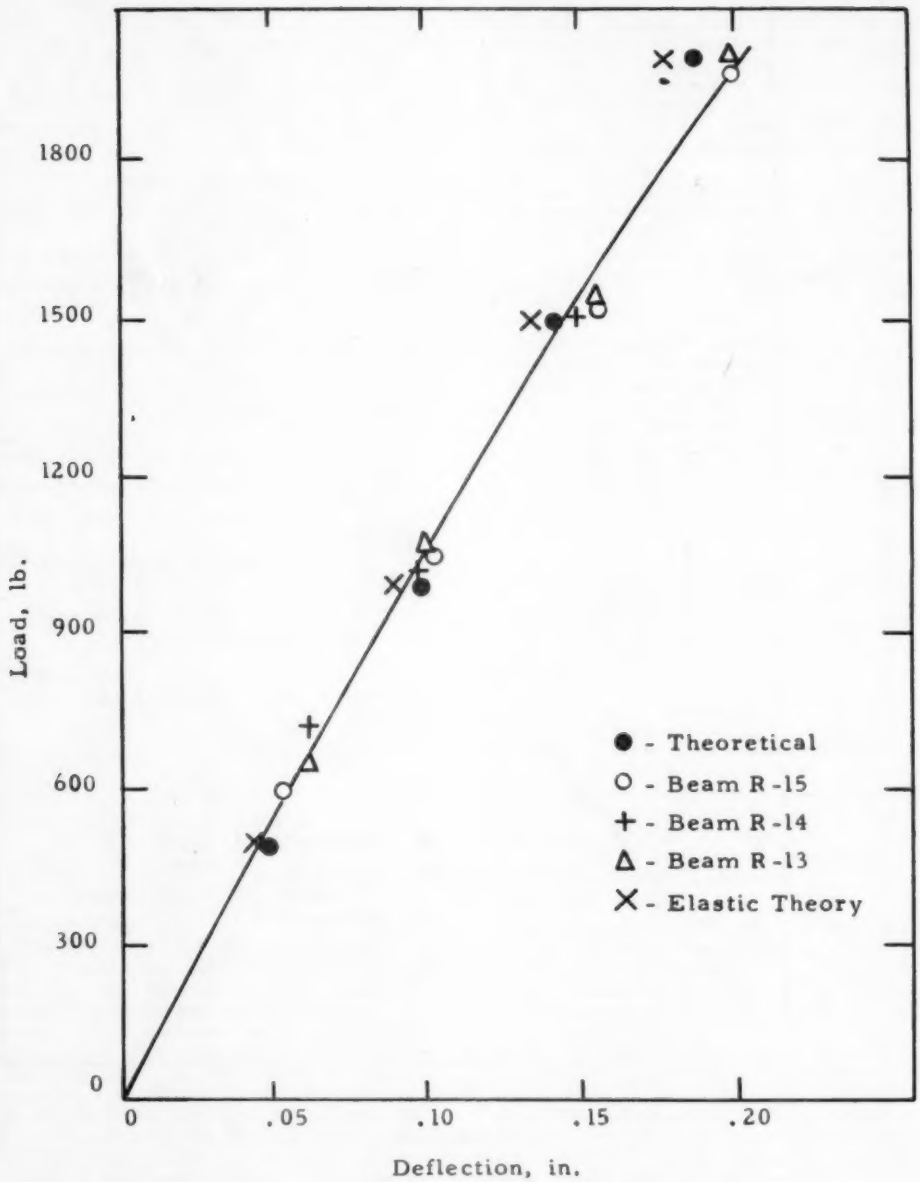


Fig. 14.--Comparison of theoretical and measured load-deflection curve, Beam R-13, R-14, and R-15

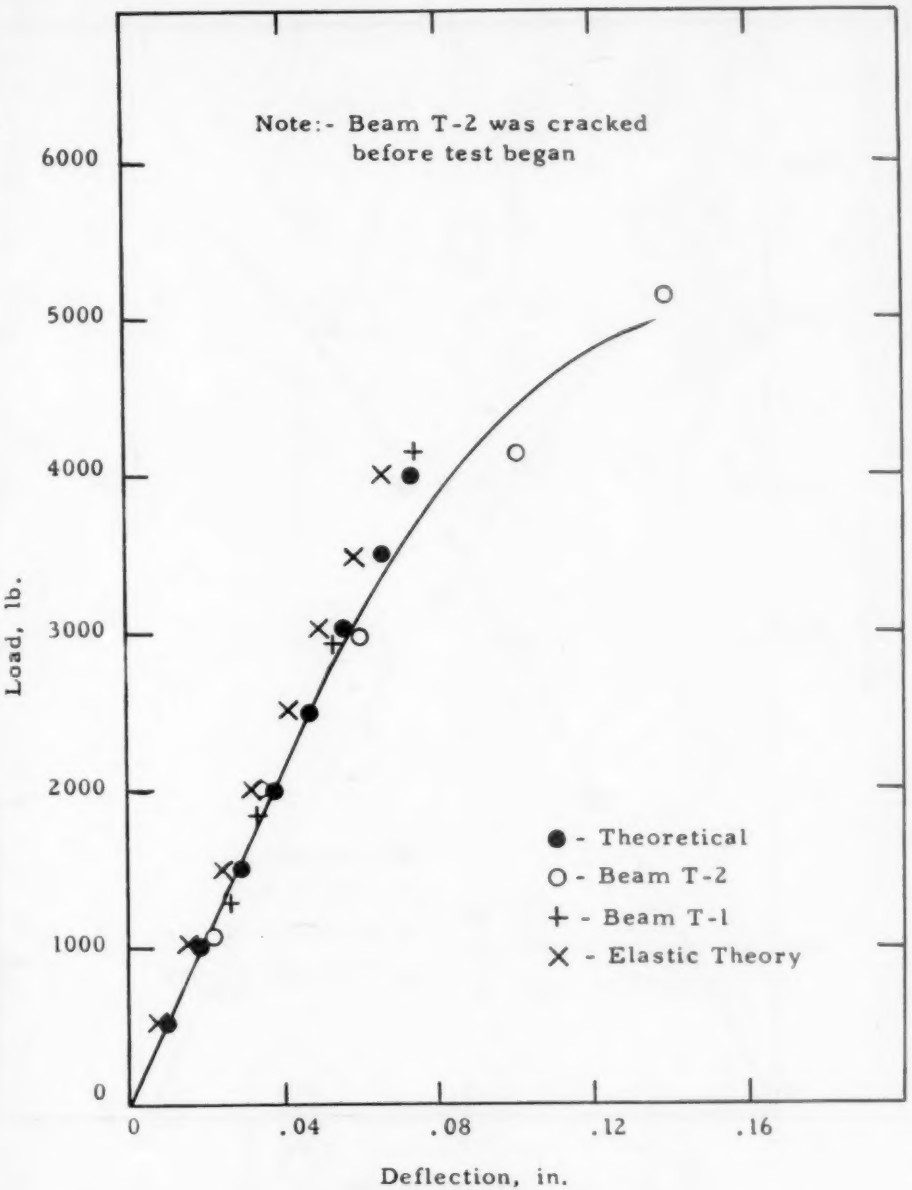


Fig. 15. --Comparison of theoretical and measured load-deflection curve, Beam T-1 and T-2

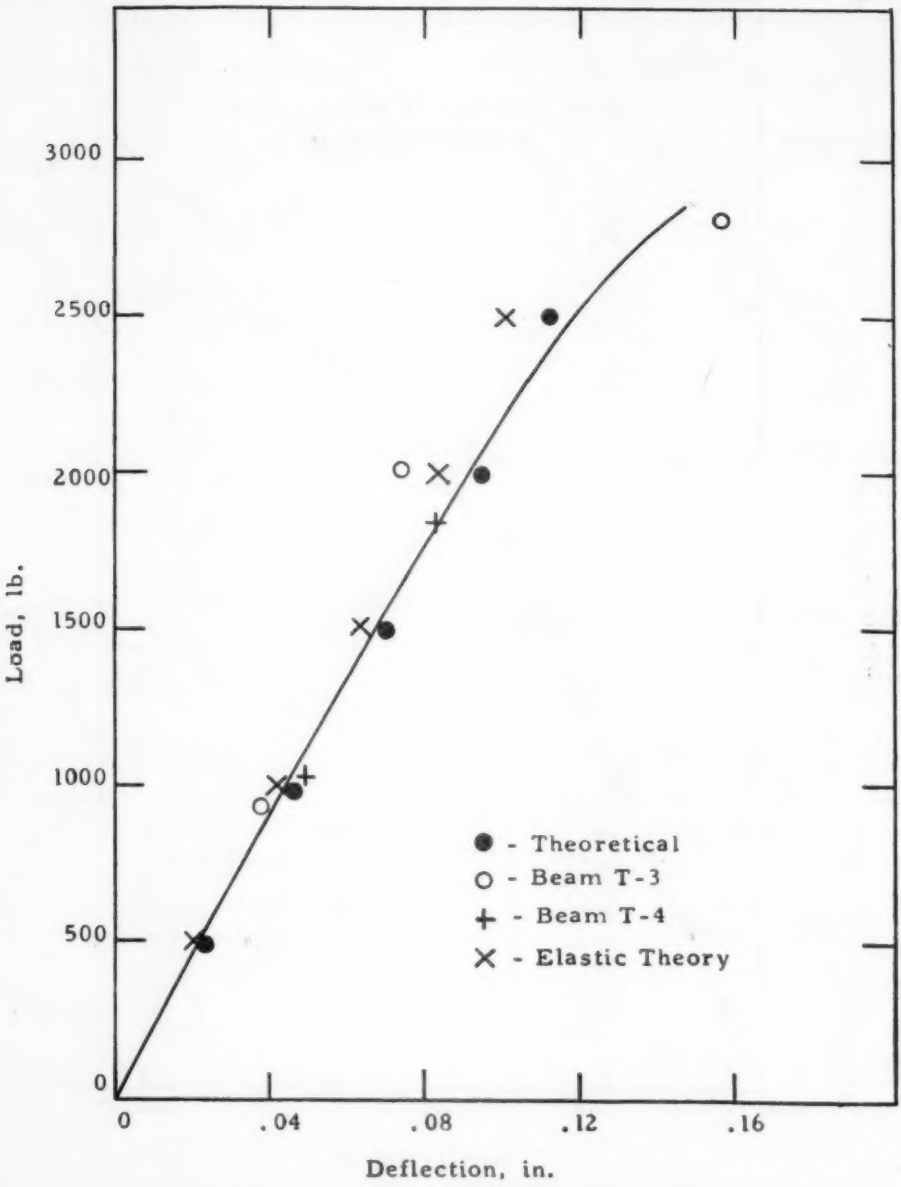


Fig. 16.--Comparison of theoretical and measured load-deflection curve, Beam T-3 and T-4

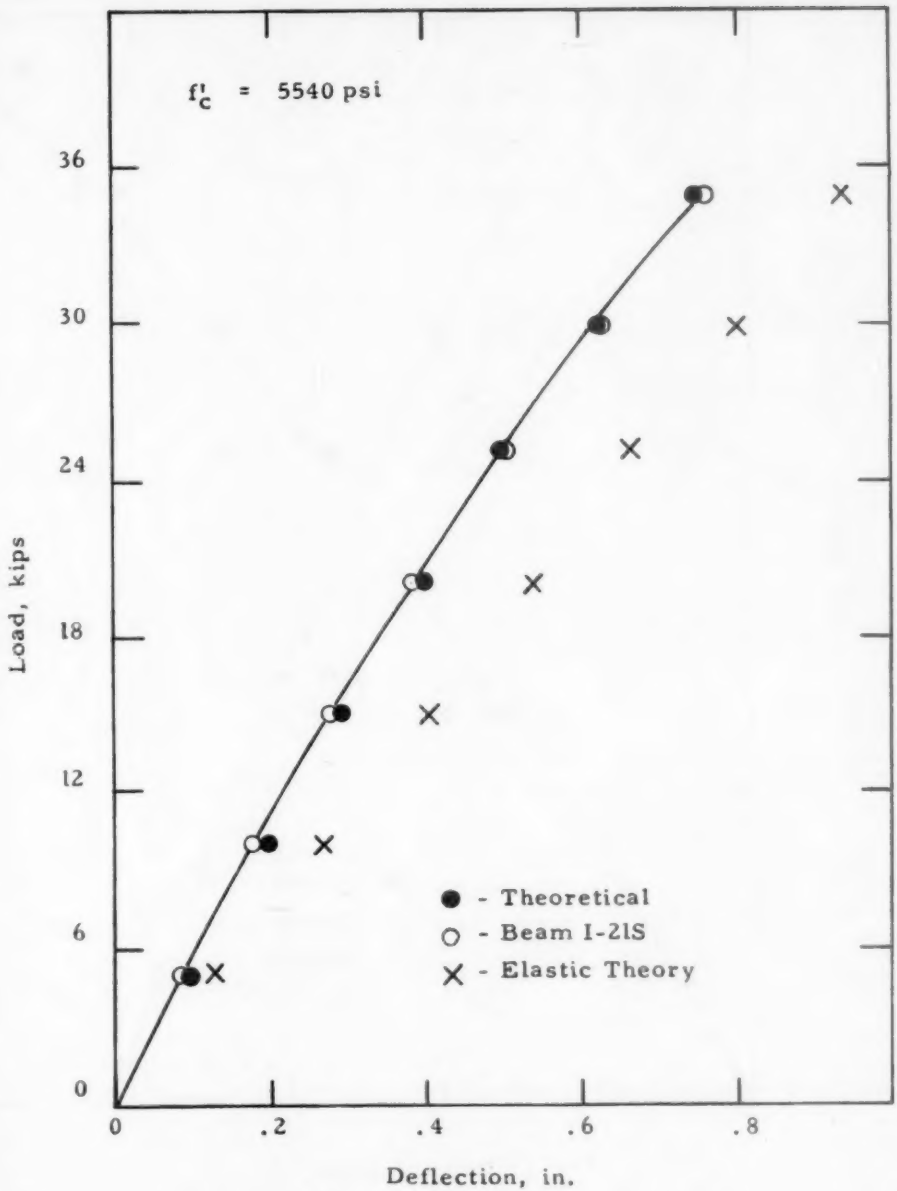


Fig. 17. --Comparison of theoretical and measured load-deflection curve, Beam I-21S

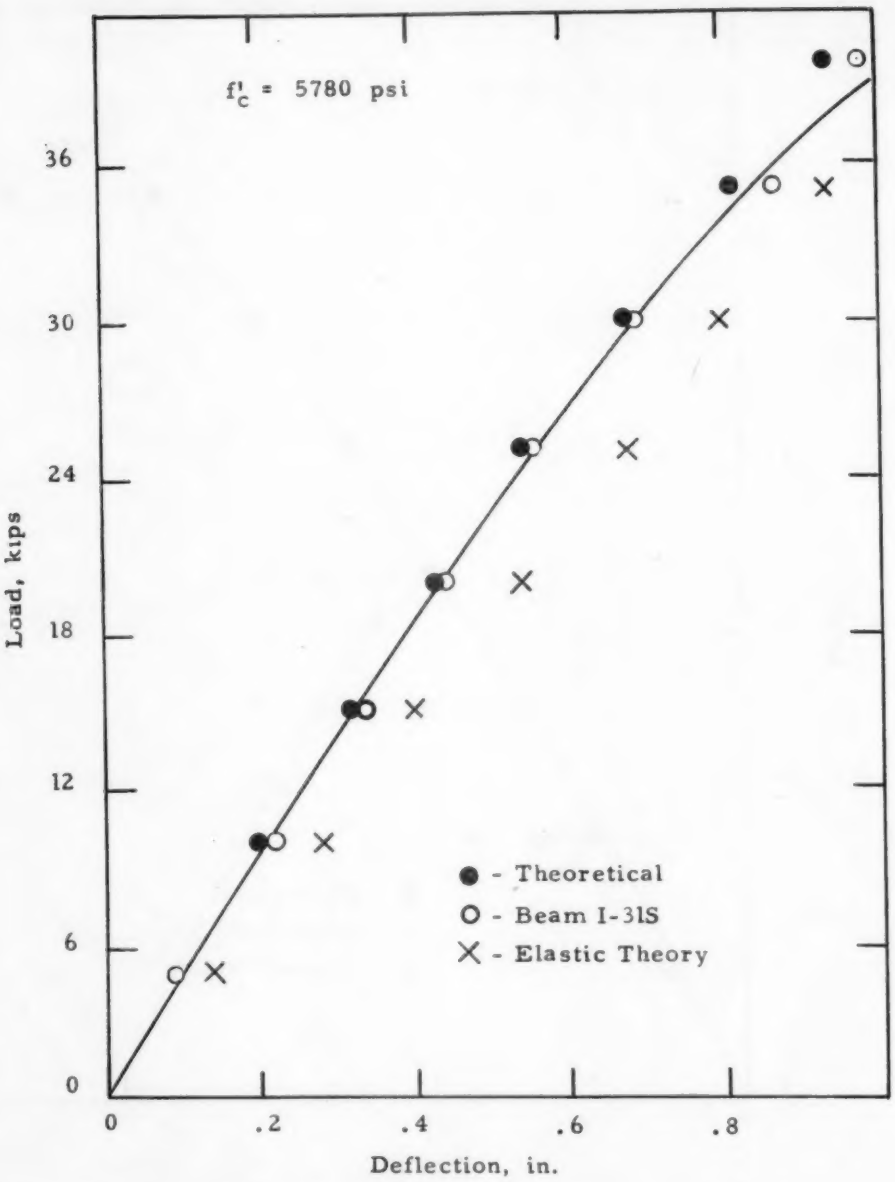


Fig. 18.--Comparison of theoretical and measured load-deflection curve, Beam I-31S

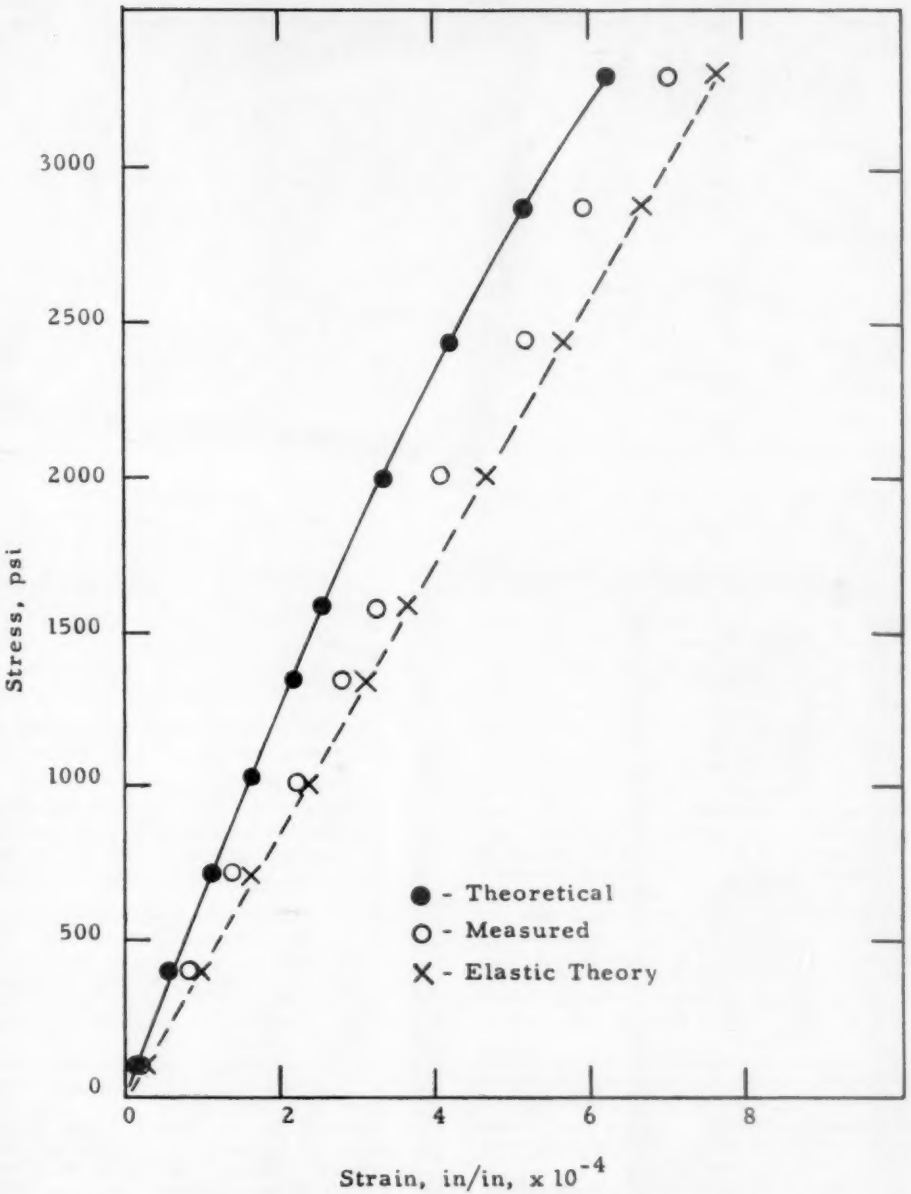


Fig. 19. -Comparison of theoretical and measured stress-strain relation, average of four beams

TABLE I
STRAIN, IN/IN, X 10^{-5}

Beam	Load Kips	Elastic Theory		Developed Theory		Measured	
		Top Fiber	Bottom Fiber	Top Fiber	Fiber Bottom	Top Fiber	Bottom Fiber
I-21S	10	20.3	14.7	14.1	11.1	26	12
	15	30.5	22.1	21.9	16.3	30	19
	20	40.6	29.4	30.3	21.3	33	28
	25	50.7	36.8	39.3	26.0	50	34
	30	60.9	44.2	49.6	30.8	58	44
I-31S	10	20.4	14.8	15.4	12.1	17.5	18
	20	40.8	29.5	32.8	23.1	37	28
	25	51.0	36.9	42.2	28.2	46	37
	30	61.2	44.4	53.1	33.4	56	46
I-41-S	10	19.5	14.1	15.7	12.1	11	11
	15	29.2	21.1	24.1	17.8	25	18
	20	38.9	28.2	33.4	23.3	35	26
	25	48.7	35.2	43.0	28.6	43	31
	30	58.4	42.3	53.9	33.8	54	38
I-41-W	10	19.5	14.1	14.1	10.9	21	10
	15	29.2	21.2	21.6	16.0	26	16
	20	38.9	28.2	29.1	21.2	39	19.5
	25	48.7	35.2	39.1	25.6	42	24.5
	30	58.4	42.3	49.1	30.4	52.5	34.5

One possibility is that the prestressing force has the effect of a horizontal load acting at the end of the beam, thus inducing a column-effect so that when the prestress force is below the centroidal axis it tends to decrease the deflection, and when above, it tends to increase the deflection. This interpretation would account for the difference between the theoretical and measured stress-strain curve and would also explain the relatively large deflections exhibited by the square sections. These sections were prestressed concentrically, therefore, when the beam started deflecting, the prestress force would be above the centroidal axis and thus increase the deflection. Although the column-effect theory might explain some of the discrepancies in the test results, it would be extremely difficult to determine quantitatively its effect on the deflection. Although the prestressing strands are bonded throughout the entire length of the beam, the bond stresses are developed at the ends and theoretically there are no bond stresses in the center portion of the beam. With this condition it seems logical that a column-effect would develop.

A thorough investigation of ultimate strains in flexure would be extremely beneficial in determining the validity of the developed theory and would also lead to a much better understanding of the behavior of prestressed concrete members in flexure.

Apparently the cross-section has no direct effect on deflections, since the developed theory gave reasonable results for all three types of cross-sections studied. The cross-section would affect the deflection indirectly of course, since the extreme fiber stresses are dependent on the location of the centroidal axis, which in turn depends on the shape of the cross-section.

CONCLUSIONS

1. The deflection of a prestressed concrete beam can be determined with reasonable accuracy by using an effective modulus as determined by the developed theory.
2. Calculated strains using the developed theory were, in general, slightly lower than the measured strains.
3. Theoretical strains were approximately as accurate as those determined by the elastic theory.
4. The shape of the cross-section, per se, has little direct effect on the deflections.

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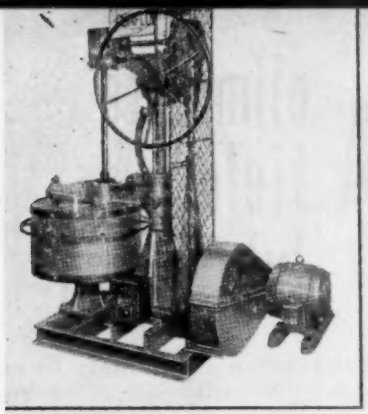

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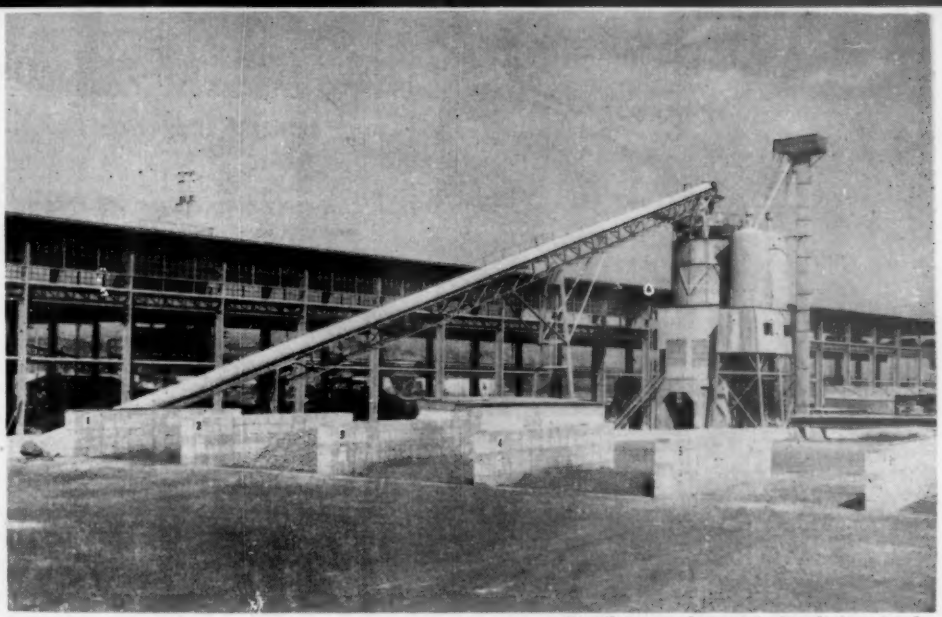

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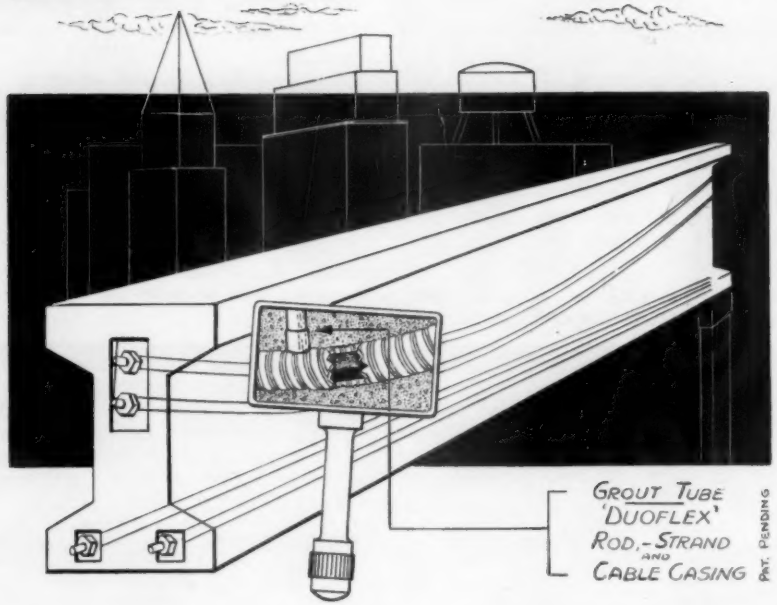

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Two types of strand are discussed in this article. At left is 7-wire incoated strand of type used for pretensioned, bonded work. At right is galvanized strand such as is used in post-tensioning, of 1-in. diameter, with 19 wires.

Wire and Strand *for* PRESTRESSED CONCRETE

H. KENT PRESTON, M. ASCE
John A. Roebling's Sons Corporation
Trenton, New Jersey

Every prestressed concrete tensioning element should have certain properties, namely:

1. High strength, to minimize the percentage of stress loss due to such factors as shrinkage and plastic flow.
2. Low coefficient of creep.
3. Ease of handling. Not only do cranky wire and strand add considerably to the contractor's costs, and therefore to his selling price, but the rough handling necessary to fight them into place often leaves them nicked, kinked, or otherwise permanently damaged.
4. Good surface for bonding to concrete on uncoated tendons.
5. Sufficient ductility, and uniform modulus of elasticity.
6. Resistance to stress-corrosion.

The wire referred to in this article is high-strength uncoated stress-relieved wire made specifically for use in prestressed concrete. The uncoated strand discussed is the 7-wire stress-relieved strand made specifically for use in pretensioned, bonded prestressed concrete. The galvanized strands for post-tensioning are the type that has been used in suspension bridges for over half a century except that the end connections have been adapted to prestressed concrete.

Wire is made by rolling steel billets into round rods and then drawing the rods through conical dies to produce the required size and strength. The drawing operation reduces the diameter and increases the length. It also changes the internal structure from the crystalline make-up of the rod to the long fibers parallel to the axis which give wire its great strength. As it emerges from the last die, the product is referred to as "cold-drawn" wire.

Wire for use in prestressed concrete is made by subjecting cold-drawn wire to a stress-relieving process. Stress-relieving is not what we usually think of as a heat treatment; the temperature used is not sufficient to change the strength of the wire but it does eliminate the internal stresses which are present in the wire as a result of the drawing process.

The uncoated strand discussed here is composed of seven cold-drawn wires fabricated by laying six wires of one size around a slightly larger center wire. In this case the finished strand is subjected to the stress-relieving operation which eliminates at one time the internal stresses due to both the drawing and the stranding operations.

UNCOATED WIRE AND 7-WIRE STRAND

Creep and relaxation. Numerous studies have been made on the subject of creep and relaxation in wire. Creep is the gradual nonelastic increase in the length of wire under a constant tension. Relaxation is a gradual loss of stress in a wire held at constant length. In a prestressed concrete member, the wire undergoes a modified form of relaxation; that is, the concrete member shortens as a result of its own shrinkage and plastic flow so that the wire is not held at a constant length but shortens with the concrete. Thus, the test values measured for relaxation in wire indicate slightly more stress loss than this factor actually causes.

Creep and relaxation properties of cold-drawn wire, stress-relieved wire, and mechanically straightened wire have been compared in as many different ways as possible. Mechanically straightened wire does not measure up — unless it is stress relieved after straightening — in which case it is in the same class with stress-relieved wire. Although these properties of stress-relieved wire are slightly better than those of cold-drawn wire in the stress ranges in which they are used, the difference is so small that the two materials can be considered as hav-

(*Reprinted from June 1956 issue of* CIVIL ENGINEERING, *published and copyrighted by the American Society of Civil Engineers*)

ing the same creep and relaxation. In considering this subject, the designer should not lose sight of the fact that a change in the density of the concrete due to more or less vibration, or a slight error in keeping to the specified water-cement ratio, can cause a change in stress loss several times any difference that can be found between the two types of wire.

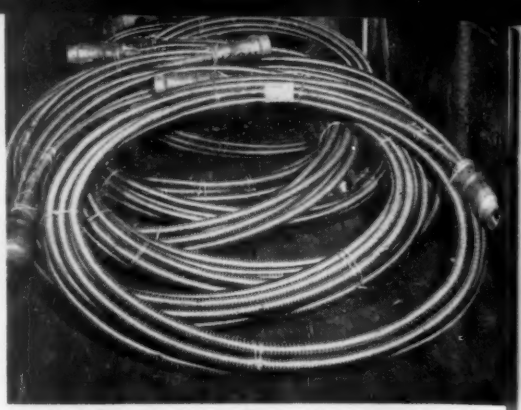
Preloading. Some designers feel that preloading wires – holding them at a tension above the specified initial tension for a few minutes before anchoring them at the specified tension – is advantageous.

It is probable that this impression stems from European practice which is explained by the report in the late Gustav Magnel's book, *Prestressed Concrete*, in which he discusses the wire used in Belgium. This wire, of 5-mm (0.196-in.) diameter has an ultimate strength of 216,000 psi. It is used at an initial tension of 123,000 psi, but its stress-strain curve begins to bend quite sharply away from a straight line at two-thirds of this stress. His tests showed a stress loss of 12 percent in the wire alone when used at an initial tension of 123,000 psi. When preloaded to 137,000 psi for two minutes, and then anchored at 123,000 psi, this loss was reduced to 4 percent. Obviously it pays to preload such a wire, especially when the preload required is only 63.5 percent of the ultimate strength of the wire.

The stress-relieved wire of 0.196-in. diameter used for prestressed concrete in the United States has an ultimate strength of 250,000 psi. Most designers use it at an initial tension of 67 percent of its ultimate. At this stress, the stress-strain curve of the stress-relieved wire is still a straight line. Tests on this wire show a maximum stress-loss of less than 4 percent from the initial 67 percent. The Bureau of Public Roads' "Criteria for Prestressed Concrete Bridges" assumes a 4 percent loss due to creep in the steel, and considers that "overstretching" is not necessary.

Tests made at Roebling's Trenton plant show less stress-loss in a wire which has been preloaded than in one which has not, but these tests cover only a relatively short period of time. DeStrycker has studied creep of wire over a period of years (*Revue de Metallurgie*, Oct. 1948, p. 411). In his tests he preloaded the wire by 10 percent for two minutes. His results indicate less loss in the preloaded wire at first, but the difference between the two procedures after several years is negligible. The results obtained by preloading stress-relieved strands are identical with those obtained by preloading stress-relieved wire.

In the final analysis, the increase in final tension obtained by preloading the high-quality wire used in the United States is so small that it cannot be applied to reduce the amount of wire in a member. On the other hand, the contractor will definitely increase his price when this operation is required in the specifications.



Galvanized strands of 1-9/16 in. diameter encased in flexible metal hose of 1-13/16 in. inside diameter, are coiled and loaded in truck for shipment. Sockets for end anchorages are tapped for permanent stud, which will be applied in the field.

Fatigue. As with any material, the stress range through which a wire can stand repeated loadings decreases as the stress increases. Using the initial stresses recommended later on, the minimum final stress in both wire and strand will be 58 to 60 percent of the ultimate. Data already available demonstrate that fatigue is not a problem on these wires and strands in this stress range. If higher stresses are contemplated, a thorough investigation of fatigue under such stresses should be conducted before they are permitted.

A review of European specifications for post-tensioned members compiled in 1952 shows initial tensions of 50 to 60 percent of the ultimate. Taking advantage of the superior elastic properties of the stress-relieved wire available in the United States, designers using this wire are specifying an initial tension of 67 percent of the ultimate. Ultimate strength varies with wire size but is in the neighborhood of 250,000 psi.

Although an initial tension of 67 percent is desirable, there are several reasons why we should not design to initial tensions in excess of 70 percent:

1. Common sense says we should not approach the ultimate too closely as there are other factors which may create stresses higher than anticipated. For instance, many cables must be over-tensioned to overcome friction between wire and concrete.
2. As the initial tension increases beyond this point, the percentage of stress loss increases much more rapidly. This means a greater differential between the initial and final tension conditions. Engineers familiar with prestressed concrete design realize that the larger this differential becomes the less efficient may be the design of the prestressed member.
3. All members must meet both a design load and an ultimate-strength specification. Excessively high stresses in the prestressing steel reduce its quantity and lower the ulti-

mate strength of the member. This often necessitates extra steel to meet the ultimate strength requirement.

4. Fatigue stresses may become a factor.

Intial tension in strand. A review of European specifications for pretensioned members compiled in 1952 shows initial tensions of 60 to 70 percent of ultimate. It should be noted that all the tensioning elements were single wires, which cannot be used in the United States for economic reasons—because of the high labor cost of handling so many small units and the need to use small aggregate so that the concrete will flow around the many tensioning elements.

An initial tension of 70 percent of the ultimate is recommended for 7-wire stress-relieved prestressed concrete strands used in pretensioned bonded members. Ultimate strength varies with strand size but is in the neighborhood of 250,000 psi. In addition to the four reasons mentioned above, there are several other reasons for using the 70 percent value:

1. The stress loss in a pretensioned member is greater than in a post-tensioned member because all the elastic compression and some of the shrinkage are eliminated by post-tensioning but not by pretensioning. The final stresses resulting from initial tensions of 67 percent for post-tensioned members and 70 percent for pretensioned members are about the same.

2. Initial tension can be specified as high as it is safe to tension the strand since it will never be necessary to over-stress the strand to overcome friction or for any other reason.

Stress-relieved materials. From the designer's point of view, stress-relieved wire and strand have several advantages over the cold-drawn materials. For instance, they have several times as much resistance to stress-corrosion; the stress-relieving process removes the wire-drawing lubricant, providing a better bonding surface; and the stress-relieved materials have a much more uniform modulus of elasticity. When a group of such wires or strands is tensioned at one time, the stress in each will be the same and the elongation can be used to check the load. Finally, the cold-drawn materials are cranky and hard to handle. This results in rough handling by the erectors and the material is sometimes damaged by the time it is in place.

All prestressed concrete fabricators in the United States insist on stress-relieved wire

Long-stroke center-hole ram tensions strand of 1-9/16 in. diameter in post-tensioned pile cap of Pier A, Hoboken, N. J. Note New York skyline in background.

because of the difficulty of handling cold-drawn wire. If cold-drawn wire were specified, the fabricators' cost would increase considerably (if they felt they could do the job at all) because of the extreme difficulty of handling this material.

GALVANIZED STRAND

Galvanized strands prove most economical where the tensioning elements are long, where the prestressing force is large, or where rust or corrosion is a consideration.

Wire for these strands is made by passing high-strength cold-drawn wire through a bath of molten zinc. The result is a thoroughly stress-relieved wire protected by an outer coating of pure zinc, plus a hard zinc-steel alloy between the zinc and the wire. Although the zinc is a protective coating which carries no load, it is standard practice in the wire industry to refer to the overall diameter of the wire including the zinc and to use the gross cross-sectional area including the zinc. On this basis the minimum ultimate strength of the wires is 220,000 to 225,000 psi, although the strength of the steel, which is carrying all the load, is appreciably higher.

Galvanized strands for prestressed concrete are made by laying one or more layers of wire around a center wire. Most of the wires are of 0.196-in. diameter, supplemented by a few wires of other sizes to get the required overall diameter and strength. Standard sizes range from 0.600 to 1-11/16 in. diameter, with minimum ultimate strengths varying from 46,000 lb. to 352,000 lb. per strand.

Although the individual wires have been stress-relieved by their passage through the molten zinc, the finished strand as it comes from the stranding machine does not necessarily have a constant modulus of elasticity. It is given a constant modulus by a prestretching operation in which it is held at a high stress for a period of time during which the individual wires adjust themselves to each other.

End anchors, referred to as fittings, are attached at the manufacturer's plant so that the strands arrive on the job ready to be placed in the structure. Several styles of fittings are available. One has a permanent stud the length of which can be varied to suit the details of the particular job. A jacking unit is attached to the end of the stud for the tensioning operation, and a permanent nut on the stud holds the load after jacking is completed. If it is in the way, that part of the stud outside the nut can be burned off after tensioning has been completed.

Another type of fitting is threaded on the outside and has a large nut to hold the load. It is also tapped to receive a jacking rod which is removed after the tensioning operation. The thread on both types is long enough to allow for reasonable errors in the length of the concrete member. A third type of fitting is a steel casting with a wide flange on the end which serves as a bearing plate. For most strands this fitting is cheaper than one with threads, and it eliminates the need for a bearing plate. The concrete is cast directly around this fitting and the tensioning is done from the opposite end, which is provided with a threaded fitting for the purpose.

Galvanized strands are used in several ways for post-tensioned members — in the open space in a concrete hollow-box, on the outside of a concrete member, or encased in a paper or plastic tube to prevent bond and covered by the concrete as it is poured. When grouting is required, strands are supplied encased in flexible metal hose, which is filled with grout after the jacking operation.

Creep and relaxation. The percentage of stress loss due to relaxation in a galvanized strand is approximately the same as that in a parallel wire cable. The initial stress is lower—140,000 psi compared to 167,500 psi —but the modulus of elasticity is also lower in almost the same ratio—25 million psi to 29 million psi—so that the two factors cancel each other.

Fatigue. Galvanized strand assemblies with fittings attached have been subjected to fatigue loadings as separate assemblies, in prestressed concrete test members, and in service in forging hammer foundations. There has never been any evidence of damage to the assembly due to such loadings. These strands are generally designed for an initial tension of 140,000 psi.

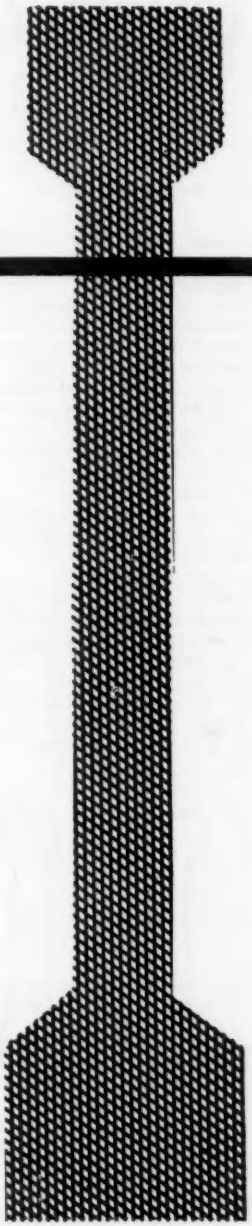
Desirable properties. Galvanized strands have several desirable properties which should be considered when the tensioning elements for a project are being selected:

1. A minimum amount of field labor is required for placing and tensioning. Regular job personnel can handle both placing and tensioning without special supervision. Long lengths are as easy to handle as short lengths because they can be supplied on reels.

2. The corrosion problem is virtually eliminated. A galvanized suspender recently removed from the Brooklyn Bridge for test after sixty years of service was found to be in satisfactory working condition.

3. Galvanized strands can be left exposed to the weather where uncoated tendons have to be protected. If grouting after tensioning is required, and the tendons will first be exposed to the weather, a galvanized flexible metal hose can be used and the entire assembly will be rustproof.

4. Galvanized strands can be tensioned more accurately to specification requirements than uncoated tendons. The coefficient of friction for the galvanized strand is much lower and quite uniform, whereas this value for uncoated tendons becomes increasingly higher as surface rust develops during the curing period of the concrete. This is of extreme importance in long or continuous structures, since high and non-uniform friction values result in varying tension throughout the length of the tendon. Galvanized strands assure much greater uniformity in this respect.



Basic Characteristics Of Cross Sections

By W. H. CONNALLY
Structural Engineer, S. Sudbury, Mass.

POST-TENSIONING

The essence of the design of post-tensioned prestressed concrete shapes can be stated in terms of three basic principles. These principles are not involved and can be demonstrated readily. They will, however, be presented here in a descriptive manner so that the reader may gain an over-all picture before embarking on a typical design problem.

The first principle is quite simple and states that the minimum size of a prestressed concrete member is determined by section modulus requirements in much the same manner as for timber or structural steel. With prestressed concrete the minimum section moduli required for the top and bottom fibers will usually be different. This results from prestress losses and different allowable stresses. Let us assume for the sake of argument that the bottom fiber ordinarily requires a somewhat larger section modulus than the top. Now consider the series of shapes shown in fig. 1. They represent members of constant depth each of which furnishes a slightly larger section modulus at the bottom fiber than at the top. From left to right they supply progressively smaller amounts of minimum section moduli. If we were to assume that they were all, say, 30 inches deep and spanned 50 feet, shape 1 might carry a superimposed load of a certain value, shape 2 a somewhat smaller amount and so on. Shape 5 represents the theoretical limiting condition at which the web alone is sufficient at the top fiber.

If we were to repeat this operation for a span length of 150 ft. and a depth of, say, 80 inches, a similar progression shown in fig. 2 might result. This time, however, the centroid of the prestressing tendons would probably fall beyond the confines of the section. The explanation for this is found in our second principle, which states

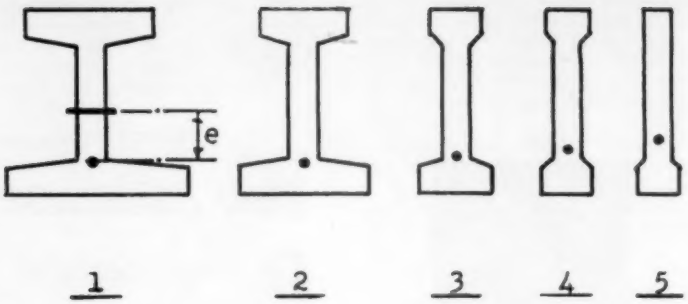


FIGURE 1 – Short span members – eccentricity acceptable

that the eccentricity for the shape supplying the minimum section moduli is composed of two items, one approximates the kern distance and the other is a function of the span length. For shapes 1 through 5 the eccentricity is not excessive because the span length is not great. The prestressing force gradually moves toward the centroid proceeding from shapes 1 to 5 because the kern component of the eccentricity decreases, approaching its minimum value of ⅓ h at shape 5. The span length component is constant for each shape. With shapes 6 through 10 this process repeats itself, but the span length component is so great that the eccentricity cannot be realized within the confines of any of the sections. Shapes 2, 4, 7 and 9, repeated in fig. 3, illustrate these points further.

The problem of the large eccentricity can be solved by our third and last principle which states that if a shape furnishes excessive section modulus at either fiber, a range of values for the prestressing force and its eccentricity is created. By inference, of course, only one possible combination can be used when the exact section moduli are supplied.

Thus we are equipped, in a general way for the moment, to discuss the highlights of post-tensioned prestressed concrete shapes.

Consider shape 2. This shape furnishes the exact value of the minimum section moduli at both fibers. It fully utilizes all its allowable stresses and therefore employs the smallest possible concrete area for its depth and for the particular flange and web thicknesses assumed. Obviously, many other configurations of the same depth can be used so long as they supply at least the minimum section moduli at both fibers. Practical considerations might dictate a symmetrical section or a member with its wider flange uppermost, shapes 2A and 2B in fig. 4. Both these shapes would of necessity furnish excess section modulus at the top fiber, although the bottom could be kept to the minimum. It is also possible that a section having a wide bottom flange might be required, shape 2C. In this instance, the modulus for

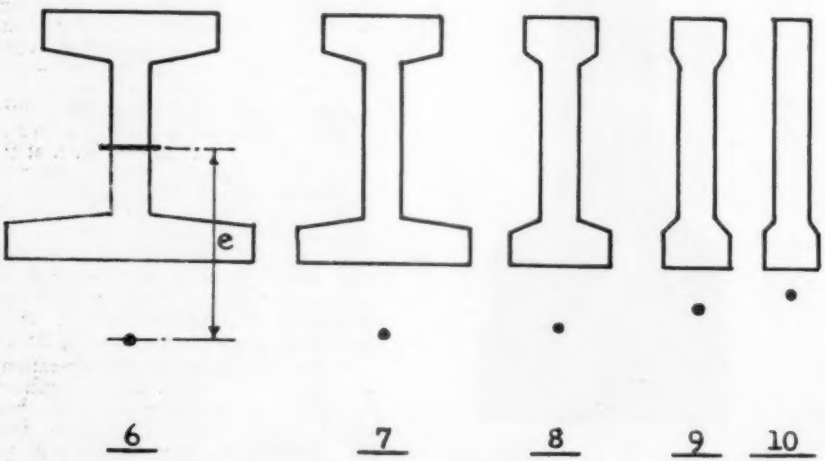


FIGURE 2 – Long span members – eccentricity too great

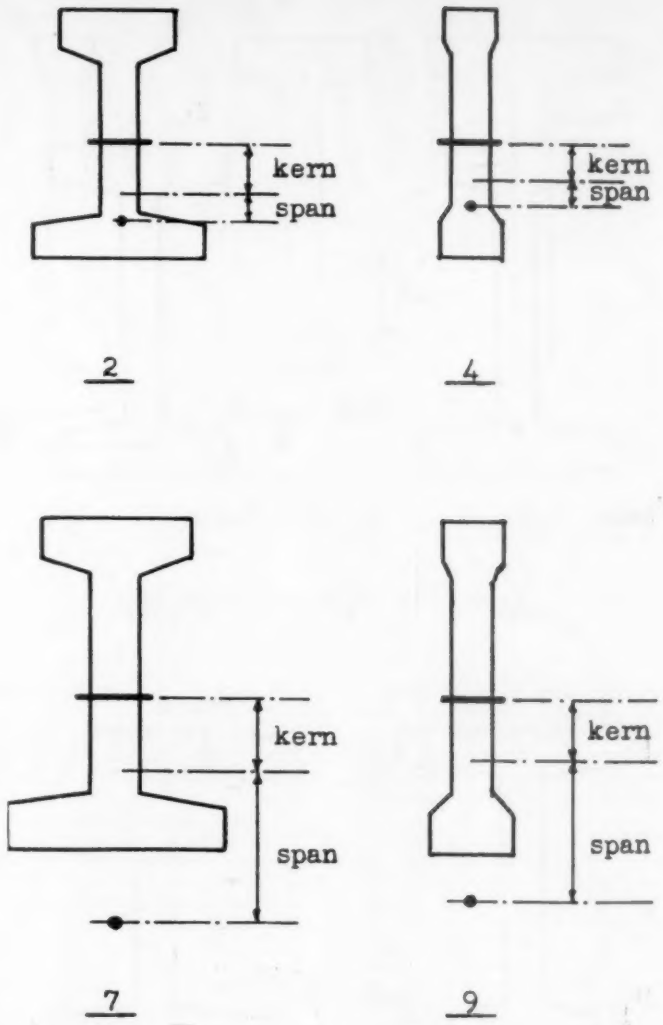


FIGURE 3 – Eccentricity components for shapes supplying the minimum section moduli

the bottom fiber would be excessive while that for the top could be reduced to the minimum.

Shapes 2A, 2B, and 2C, because they supply excessive section moduli, allow a range of acceptable values for the prestressing force and its eccentricity. The extent of this range for the eccentricity has been indicated by Δe in fig. 4; its actual mathematical extent can be defined by a design equation. The designer will select the most economical value of e that can be obtained within this range. That value farthest from the neutral axis will result in the smallest prestressing force. The latter, of course, can be positioned no closer to the bottom fiber than fire protective requirements, minimum spacing requirements, and the physical make-up of the prestressing system. Sometimes Δe will be such that the prestressing force can be located at the minimum allowable distance from the bottom fiber as arbitrarily indicated for shapes 2B and 2C. On the other hand, the extreme limit of Δe may control as shown for shape 2A.

While shape 2 employs the smallest concrete area, shapes 2A and 2B may allow a reduction of the prestressing force. The amount of this reduction will not usually effect too great a change in the actual steel areas used, however. This point is illustrated by Examples 7 and 8 in Section 3. Under certain conditions, even a shape like 2C, with excess section modulus at the bottom

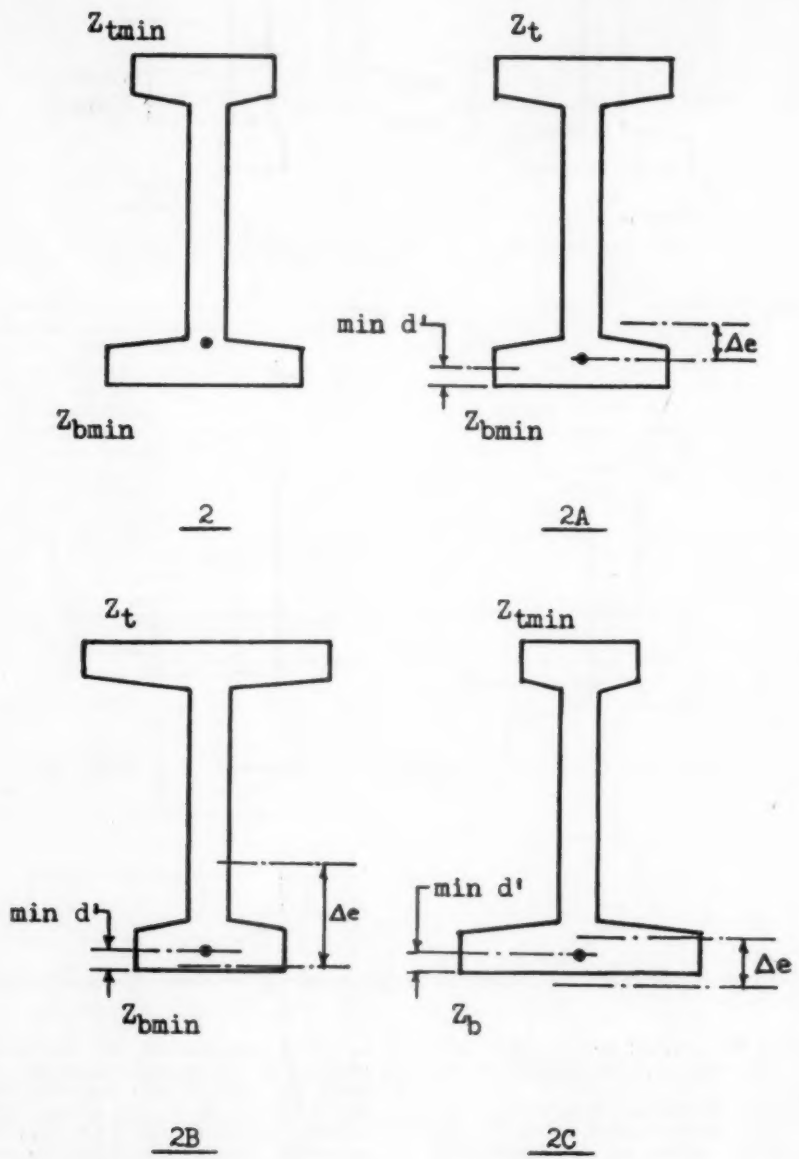


FIGURE 4 – Range of eccentricities for shapes supplying more than the minimum section moduli

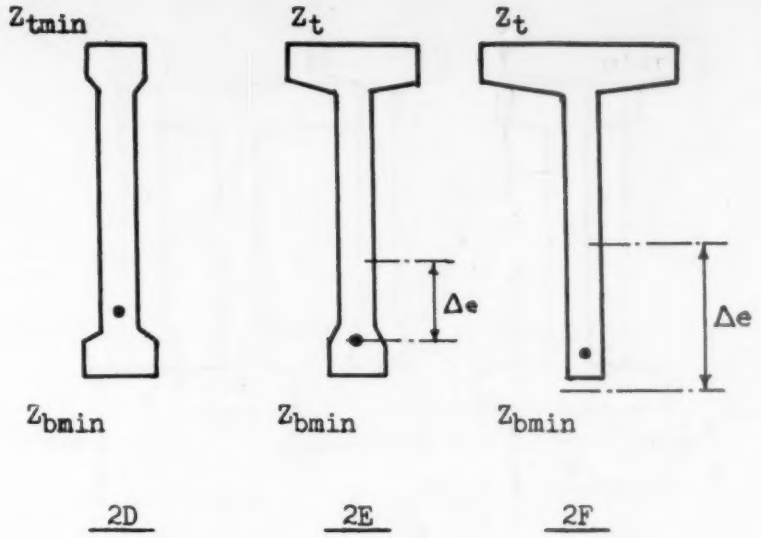


FIGURE 5 – T-Beam from a large depth

fiber, may show a slight reduction in the prestressing force.

Also of interest is the possibility of using a T-Beam in place of shape 2. If we were to increase the depth of this section, still holding to the minimum section moduli, the flange widths would gradually diminish, resulting in, say, shape 2D, fig. 5. This shape can be expeditiously converted to a T-Beam by adding increments of section modulus to the top fiber while holding the bottom to the minimum, eventually yielding shape 2F. It would have been impossible to achieve this end with shape 2 because reduction of the relatively wide bottom flange would have resulted in a top flange of greatly prohibitive width.

Consider now shape 4 in fig. 1. This member carries a much lighter load than shape 2. The latter might represent beams spaced at 20 ft. intervals, and the former joists at a 6 ft. spacing. All of the previous discussion for shape 2 applies for this case. A symmetrical section might be employed as well as a member with its wider flange uppermost or vice versa. As regards a T-Beam, the shape as it stands can be readily converted to one as is apparent from the discussion of shapes 2D and 2F. Obviously, the conditions favorable for T-Beams are small bottom flanges. The latter evidently result from large depths or light superimposed loads. These points are illustrated by Examples 11 and 12 in Section 3.

Let us now consider shape 7. This section has a configuration similar to shape 2 although it is much deeper. Because of its long span, we find, in accordance with our second principle, that the prestressing force falls outside the limits of the section. Assuming that our final solution will have the same depth as shape 7, we have two possible alternatives. We may increase the section modulus at the top flange, keeping the bottom to the minimum until Δe is great enough to allow an acceptable solution; or we may do the opposite and provide excess section modulus at the bottom. These alternatives are illustrated by shapes 7A and 7B in fig. 6. Note that shape 7B does not allow a solution. In general, for the conditions normally encountered in practice, the provision of excess section modulus at the bottom fiber will aggravate rather than relieve the initial condition of an impractical eccentricity.

If still more section modulus is provided at the top fiber for shape 7A, the prestressing force may reduce slightly. The amount for this reduction will usually be unimportant.

Let us now turn to shape 9 in fig. 2. This member is similar to shape 4 except for the location of the prestressing force. The most favorable solution will be found by gradually adding increments of section modulus to the top fiber until Δe allows an acceptable solution. The bottom fiber can, of course, be kept to the minimum. If the amount of excess Zt required is great enough, this can again result in a T-Beam as arbitrarily indicated in fig. 7.

It might be argued at this point that in addition to large depths and light superimposed loads, T-Beams also result from long spans. This statement is valid but only for

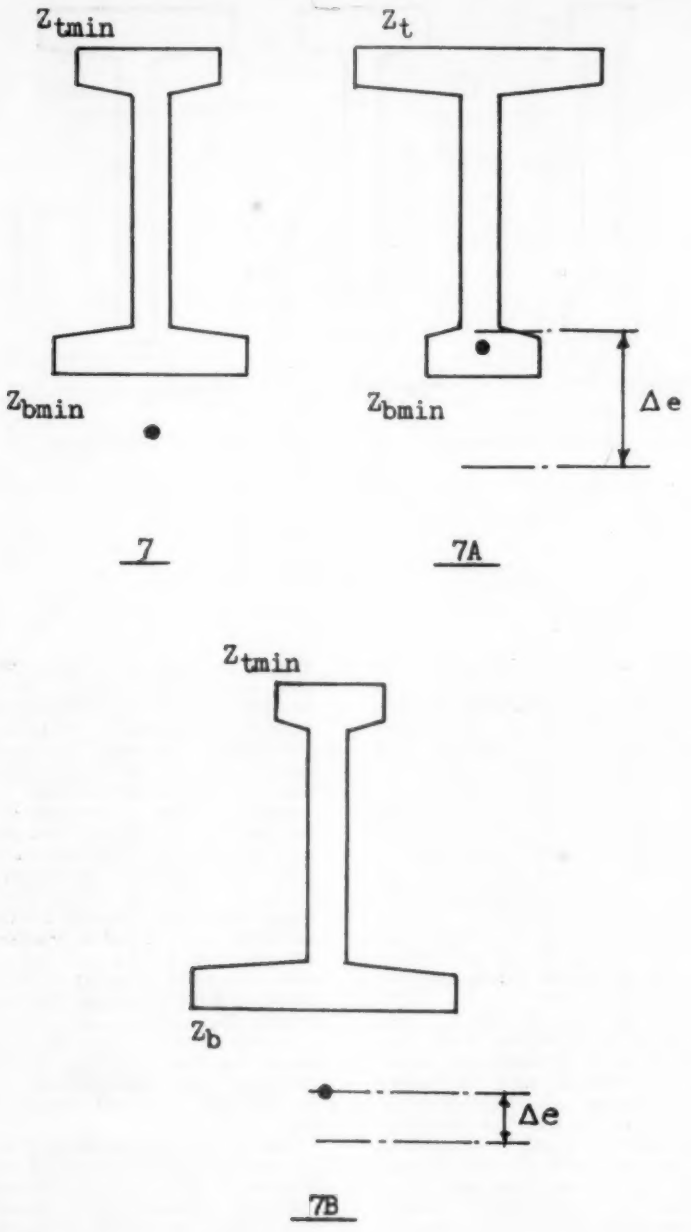


FIGURE 6 – Acceptable solution for long span member

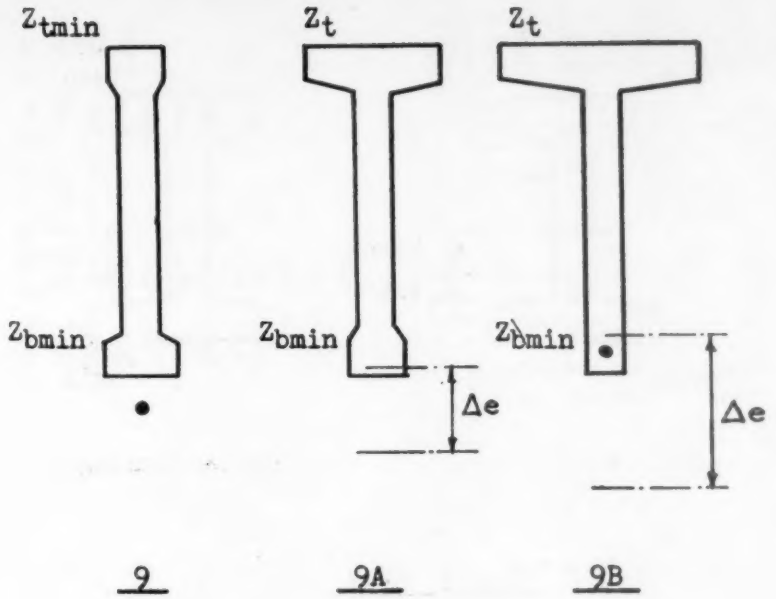


FIGURE 7 — T-Beam from a long span

special conditions. The width of the bottom flange must be small enough initially (which implies a large depth or a light superimposed load) and the span length great enough so that the amount of excess Zt required will be of the proper magnitude to cause bb to diminish to b' before a practicable eccentricity becomes available.

PRETENSIONING

With pretensioning a series of shapes corresponding to those shown in figs. 1 and 2 for post-tensioning also exists. In each case, however, the required minimum section moduli are larger. As will be explained, this results from the fact that the minimum section moduli are determined by the live load plus the entire dead load — while for post-tensioning the live load plus only a fraction of the dead load governs.

The eccentricity for the shape supplying the minimum section moduli has only one component as opposed to two for post-tensioning. With pretensioning the span length component does not exist and the kern component alone controls. Because of this the minimum eccentricity can always be realized even with long spans. The concrete and steel quantities required, however, will be greater than for a comparable design in post-tensioning. These points are illustrated in fig. 8. On the short span the dead load is relatively small compared to the superimposed load, and the difference between the two sections is not great. On the long span the dead load represents a much greater proportion of the superimposed load and results in a pronounced increase in size for the pretensioned member. Of course the top flange of the post-tensioned member will have to be increased to yield a workable solution, but the final shape will still employ less steel and concrete areas than the pretensioned section.

As with post-tensioning, the provision of excess section modulus at either fiber will allow a range of values for the prestressing force and its eccentricity. The maximum extent of this range will be at or near the kern regardless of the amount of excess section modulus supplied.

All of the above statements apply only when the wires or strands are carried straight through the member, and when the member itself is straight and of constant cross section. This automatically positions the prestressing force at or near the kern because of conditions at the end of the span. If the cross section is varied, or the wires displaced, or both halves of the member sloped, then it may be possible to position the wires as desired in the middle of the span and still be near the kern at the support. In such cases the basic design characteristics at the critical section will be the same as those discussed for post-tensioning. This will also be true for straight members with straight wires if the prestressing force is positioned beyond the kern, close to the bottom fiber. Of course, adequate provision must then be made at the ends of the member to counteract the tensile stresses induced in the top flange.

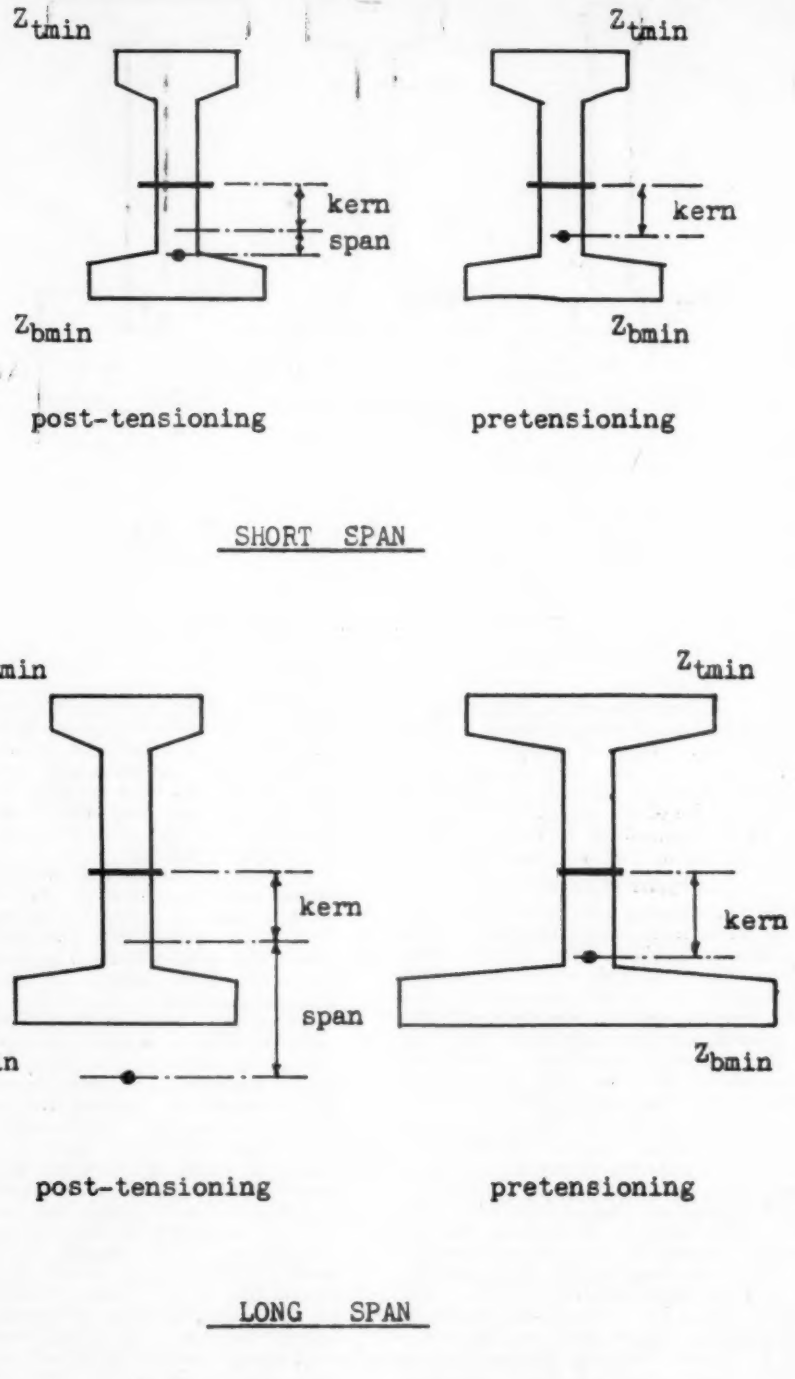


FIGURE 8 – Pretensioning versus post-tensioning

The Future Of Prestressed Concrete

By BEN C. GERWICK, JR.

Presented
to the
25th Annual Convention of the
Structural Engineers Association
of
California
on
October 13, 1956
at
Reno, Nevada

When the Chairman of your Technical Program Committee, Mr. Leonard Hollister, asked me to make a presentation on prestressed concrete at this convention I was naturally very pleased and highly honored. A talk on prestressed concrete for 30-45 minutes would not be difficult.

However, when he subsequently asked that I point this presentation along the theme of this convention, "The Next Twenty-five Years in Structural Engineering," by taking a look at the next twenty-five years of prestressed concrete, I began to realize the magnitude of the task before me.

The progress made by prestressed concrete has been quite rapid. Its increasing use during the last three years, which is the period of its greatest growth, has been phenomenal. It will indeed be a most satisfying experience if the next twenty-five years in prestressed concrete are as progressive, interesting and stimulating as the last three years have been. It is truly an exciting material with unlimited possibilities for the future.

Whatever I say about the next twenty-five years of prestressed concrete I feel that the facts of 1981 will perhaps prove me to be a short-sighted prophet. However, I will attempt to comply with Mr. Hollister's request by looking as far forward as present knowledge permits.

In order to better understand the part which prestressed concrete will play in our future work, let me review some of the reasons and forces which will be the cause of its continued growth.

THE FOUNDATIONS FOR ANTICIPATED GROWTH OF PRESTRESSED CONCRETE

Actually, the development and application of linear prestressed concrete in the United States dates only from 1950. From one plant and one bridge in 1950, we now have throughout the United States close to 150 prestressed concrete plants. Prestressed concrete foundations and structures are beyond count. Most of this growth and application, as I mentioned, has taken place in the last three years.

The greatest period of growth of any industry will, of course, normally take place during the early stages of its development. Measured in terms of years, as compared to structural steel, reinforced concrete and timber, prestressed concrete is still in the early development stage. If we project this tremendous early growth over the next twenty-five years it becomes evident that prestressed concrete is destined to become a major structural material in building toward the fabulous future which business economists, labor leaders and politicians have predicted for us.

As engineers, we accept and use materials for one or both of two reasons – the material is either a better product structurally for a particular job, or it is more economical. In our economic system any manufacturing industry must stand or fall on meeting either or both of the above criteria. Based upon proven experience throughout the country prestressed concrete will, when properly used, meet these criteria. Exactly how and to what extent I will cover later on.

However. there are certain forces, economic and sociological, which we must consider if we are to gain the proper perspective of the future of the prestressed concrete industry and the part it will play in the construction industry.

The key to the goals of 1980 which have been established for us by planners and visionaries is the increased productivity of the individual worker. If we are to have more goods, both as individuals and as a nation, then we must be able to produce more as individuals. However, in your thinking and planning for the future do not overlook the fact that, while we must have increased productivity, labor and business is thinking and planning for the 35-hour, then the 30-hour week.

Therefore, the challenge which we, as engineers, must meet in the future is this: We must not only turn out more work than we are doing today, in 12% to 25% less time, but we must design projects which can be constructed with a corresponding savings in time. Such demands on our abilities must be satisfied if we are to keep pace with the great changes taking place in our country today. The course before us is dictated by necessity, not by choice.

We therefore add a new criteria for prestressed concrete – and for any material – to those of structural and economic requirements – that of time. Although the factor of time is interwoven with economics it should be considered separately in many instances. So, we must learn to do more and build more with better materials – and in less time – than we have today. There have been many notable developments in building technology in recent years which have contributed toward that end. Curtain walls, precast thin shells, lift-slabs for roof and floor, slipforms, precast panels and many other innovations indicate the trend to structures with a maximum of standard time saving products and techniques. It is true, as has been recently said, that in prefabrication we have the beginnings of the industrialization of the construction industry.

In prestressed concrete we have an extension to this trend of prefabrication. Whether the project be a bridge, building, marine project or piling, the prefabrication advantages offered by precast prestressed concrete are these: A standard product of great structural strength, of known and tested quality, manufactured to exact tolerances under close supervision for quick and efficient erection. While these advantages will always exist the economy of prestressed concrete will be determined only upon an analysis of the problem and the requirements for the particular project in question.

The prestressed concrete industry throughout the country is attempting to give you standard sections for your projects. We want you to be able to go to a catalogue of prestressed concrete products, pick out a section or sections for your project in the same manner as you do steel members, check it out and include it in your design by number, the same as you do steel.

As many of you know, we recently issued catalogues on prestressed concrete piles and building members. It is gratifying to report that our standard pile designs have been specified on numerous projects from the deep South to the West Coast, Alaska and the Philippines. Three recent local buildings have been designed using the standard sections out of our catalogue. Except in special cases there is no reason why you should have to design prestressed concrete members. As a manufactured product it is up to the prestressed concrete industry to provide you with standard designs and data for your consideration and use. Such standardization will lead to greater economy in your design costs and, from the manufacturing point of view, production costs.

Remember too, that precast prestressed concrete products will effect a savings in construction time. Instead of construction progressing chronologically, the use of precast prestressed concrete, as well as other prefabricated units, will permit several operations to be conducted simultaneously with a subsequent tying together of such units at the site.

In prefabrication of structural elements at centralized plants it is possible to increase the productivity of the individual to a considerable degree. This can be accomplished by factory line production methods with a maximum utilization of labor and time saving devices. For work which must be done at the site the answer has become mechanization. The pattern of the future is therefore established – prefabrication and / or mechanization wherever possible. It is only by an application of one or both principles to our jobs that we can meet the challenge of turning out more work in less time.

Because of this necessary trend to prefabrication in the engineering and construction industry and the ever constant search for better materials, I believe that the use application of prestressed concrete during the next twenty-five years is almost beyond prediction. I have no doubt that the prestressed concrete industry, composed of many centrally located plants, will become one of the giant industries serving the engineering and contracting professions. And, I predict that this will happen before the 50th anniversary of this Association.

TYPES OF PRESTRESSED CONCRETE

As you know, there are two types of linear prestressed concrete; in pretensioned prestressed concrete the strands are tensioned prior to pouring of the concrete with the prestress transferred to the concrete by bond. Post-tensioned prestressed concrete may be either the bonded or unbonded type, normally the former, and it is stressed after the concrete has been poured and cured.

Generally speaking the pretensioned concrete will be more economical than the post-tensioned type, as is proven by the unmistakable trend to this type of fabrication. Though capital investment for a pretensioned concrete operation is much higher than for a post-tensioned concrete operation, the cost per unit produced is usually lower. Pretensioned prestressed concrete is not a "shoestring" operation as many have found out the hard way.

A pretensioned member will generally be of more uniform quality. The reason for this is the same as applies to any industry – factory line production of a standard product at a centrally operated plant with close supervision of quality of materials and workmanship. However, precast post-tensioned members, produced at a central plant, have been economically competitive in many instances.

Today, pretensioned concrete products are always precast in a centrally operated plant while post-tensioned products may be either

Aerial photo of Ben C. Gerwick, Inc., Petaluma plant. Barge load of 20" square pretensioned piles in foreground shipped 350 miles up Pacific Coast for bridge project. Bed on left of track is 500' long; one on right is 340' long. Another 400' long bed has just been added.

precast or cast-in-place and then post-tensioned. Pretensioning of cast-in-place concrete, in the future, is quite probable on certain types of projects. Before I analyze the present uses and possible future applications of prestressed concrete let me discuss briefly the two basic materials of prestressed concrete — concrete and steel.

MATERIALS FOR PRESTRESSED CONCRETE

The basis of prestressed concrete is the proper use of the best qualities of each material — compressive strength of concrete and tensile strength of steel. While basically the contents are the same for reinforced concrete, prestressed concrete is an entirely different material in properties and characteristics and should always be considered as such and not as a refinement of reinforced concrete.

Today we use 7 or 8 sack mix concrete, either a Type II, or a cement which meets Type II specifications, in our pretensioned concrete work. Combined with good aggregate and steam curing we obtain concrete strengths of 4000 to 5000 psi in 24 to 36 hours depending on size of members, 6500 to 7000 psi in 28 days. This is normal for pretensioned concrete.

The steel we use is a 7-wire high strength uncoated stress relieved strand made specifically for pretensioned concrete work. The ultimate strength of this 7-wire strand is guaranteed 250,000 psi. The strands are tensioned to 175,000 psi, or 70% of ultimate, with a design load of 140,000 to 150,000 psi. By combining these great strengths of concrete and steel under centrally controlled conditions, the result is a product of high structural strength. It is quite likely that during the next 25 years cement will be improved so that even greater strengths can be obtained. Too, it is possible that strand strength will be increased with the same resulting effect — greater strength per strand and per member or the same strength with less strands.

At present the industry primarily uses a ⅜" dia. strand, but 7/16" is being increasingly used in various parts of the country. In California ⅜" has been used mostly, though I feel that a change to 7/16" is warranted in the near future. ½" strand is under test and is being used on some projects.

To properly evaluate the present and future uses of prestressed concrete it would be advisable, I feel, to speak on their specific application in bridges, buildings, marine projects and piling.

BRIDGES

It is probable that in dollar volume more prestressed concrete has been used in bridges than in any other type of structure.

From the first bridge in the United States in 1950 through 1952, prestressed concrete was often chosen either because of shortage of steel or because of a desire to try this new material. Since then it has generally been chosen because of its merits structurally and economically. From 30-ft. to 300-ft. spans, prestressed concrete has competed, and often won, against steel designs in all parts of the world. The most notable example is the well publicized Lake Pontchartrain Bridge in New Orleans, 24 miles long, which was bid about 20% below a steel design and completed four months ahead of schedule.

In California the following table is indicative of the growth of the use of prestressed concrete for bridges. It is quite definite that the number and tonnage of bridges in 1957 will be greater than is shown when the new Federal Highway program projects have been included.

Year	Number of Bridges	Total C.Y. Concrete	Total Weight Tons
1954	3	6,625	13,248
1955	8	3,489	6,978
1956	43	10,764	21,528
1957	18	14,845	29,690

A very important development in the use of prestressed concrete by the State of California is the publication of standard designs for precast prestressed flat slabs from 20-ft. to 50-ft. and channel sections from 30-ft. to 60-ft. The use of such standard designs will not only shorten the design time of such structures, but will enable manufacturers to amortize their form costs over a number of jobs rather than for each job, thereby leading to lower costs per unit. In addition to their use by the Division of Highways, these standard designs have been issued to all

county Road Commissioners in California, and will be of great help to them in their county bridge programs.

It is hoped that the Division of Highways will standardize sections for spans from 60-ft. to 120-ft. in the near future. Sections for such spans will probably include I, T and hollow box girders for the different situations which may be encountered.

For shorter spans our studies indicate economic advantages in flat solid slabs. Without hollow cores to place or channels to form, labor costs per unit are low. The use of flat solid slabs is controlled to a great extent by span length and distance of shipment. Their use will generally vary inversely with the length of span and shipment.

In a recent State Highway bridge project in which these flat slabs were used, there were 36 – 23′ x 5′ 10″ x 10″ units. All of them, totalling close to 5000 sq. ft., were set in one day. Trucks were using the bridge the next day, immediately after the units were transversely post-tensioned.

For long span girders and under certain loading conditions it is necessary to have the c. g. of the steel in a parabolic path, rather than straight. Up until recently it has been necessary to use the post-tensioning method of prestressing to obtain this curve. This, of course, added to the cost of the member as each one had to be tensioned individually.

The industry has recently overcome this advantage of the post-tensioned method by manufacturing bridge units with deflected or "harped" pretensioned strands with the c. g. for the steel following a parabolic path. This is an extremely important technical development toward the more economical manufacture of units which require a parabolic curve. This method of manufacture will permit better control of the tensioned elements with no friction loss or grouting problems common to post-tensioned design.

Besides the numerous prestressed concrete projects to be undertaken in the future by the California Division of Highways are two projects of considerable magnitude. The Southern Crossing of San Francisco Bay is specifically designed in pretensioned concrete. As large as this project will be, it will be dwarfed by the scope of the projected Rapid Transit System for the Bay Area. Tentative plans call for competing designs in steel and pretensioned prestressed concrete.

In prestressed concrete bridge construction the trend for the future seems to be to longer spans. We have actively studied 150-ft. spans for a railroad project and 300-ft. precast prestressed girders for an overwater highway bridge project. I have recently read of a study made in France for a 1000-ft. cast-in-place prestressed suspension span. This indicates practically no limit for cast-in-place structures.

A precast, though not wholly prestressed project which we recently completed for the Sacramento Northern-Western Pacific Railroad at Yuba City-Marysville, aptly demonstrates the time saving advantages of prefabrication. Here we designed and constructed a 1500-ft. bridge and an 80-ft. overcrossing on 40-ft. spans in about four months time. Units used were precast and prestressed concrete piles and precast concrete girders.

Most of my comments have been on highway bridges where the greatest application of prestressed concrete to date has taken place. This development will continue at a greatly accelerated rate under the impact of the recently approved Federal Highway bill. I have no doubt that prestressed concrete girders will eventually replace structural steel as the major structural material for spans up to around 100-ft. The reasons why this will happen are quite clear – prestressed concrete is available, it is economical and it is structurally equal to steel with less maintenance problems.

However, highway bridges are but one phase of the bridge field. Of great importance also are railroad bridges and trestles. Committees of the Association of American Railroads have done considerable research on the matter of prestressed concrete railroad bridges, as well as prestressed piles, building members, etc., and have approved the use of prestressed concrete in principle, with economics to be the general deciding factor in its use. All indications point to a shift from steel to precast and prestressed concrete as the basic material for railroad bridges with prestressed concrete also competing with timber trestles in many instances.

We feel that the pattern for the future of prestressed concrete bridge construction – highway and railroad – has been established. Growth will be due to availability and economic and structural competitiveness with a trend to standard sections and longer and heavier spans.

I believe this is the proper place to recognize the leader in the practical application of prestressed concrete in the State of California, the Bridge Department of the Division of Highways under Mr. F. W. Fanhorst, and his two Assistants, Mr. A. L. Elliott and Mr. I. O. Jahlstrom. They have contributed immeasurably to the development and use of precast prestressed concrete piles and bridge girders in the West through their early and continuing interest in the subject.

BUILDINGS

To date the most efficient and common use of prestressed concrete in buildings has been in low level structures such as warehouses, commercial and industrial buildings, schools, etc.

Products which have been used include beams, girders, purlins, columns, roof and floor channel, and double-T sections. The majority of such items used have been of the pretensioned variety due to their greater economy than post-tensioned products.

Many all prestressed concrete structures have been built in Florida, which is perhaps the leading State in the Union in the use of prestressed concrete for buildings. Here on

the West Coast the use of such products for buildings has been greatest in San Diego, Los Angeles, and in the Tacoma-Seattle area. In Northern California, prestressed concrete to date has been limited to a few roof structures and girders.

This lag in the use of prestressed concrete for buildings in this area is quite unusual since one of the areas in the U. S. in which there are two active competing prestressing plants is in Northern California. Elsewhere on the West Coast and in most other locations only one plant exists. However, I'm quite sure that such a situation will change soon. Many Northern California engineers have indicated their intentions of designing prestressed concrete structures in the immediate future, and some have them on the drawing boards now.

In speaking of prestressed concrete building structures I might point out that the preceding comments pertaining to a lack of use of prestressed concrete in Northern California does not apply to the field of prestressed lift slabs.

The California Division of Architecture, under Mr. Anson Boyd and his Supervising Engineers, Mr. C. M. Herd, Mr. C. Peterson and Mr. Alan Brownfield, has been a leader in this application and has several lift slab structures completed, utilizing prestressing forces in two directions. This is the most accepted method of prestressed lift slab construction and in some cases may prove to be desirable for other types of prestressed units. The Division is quite aware of the potentialities of prestressed concrete and, we are told, will have no hesitancy about using precast prestressed products whenever their use appears feasible. I believe they have two or three such projects under design now.

While there will be unusual and novel applications of prestressed concrete in buildings during the coming years, the main use for the future appears to be the same as for the present – low level structures with a maximum of duplication of members.

There are two possibilities in the use of precast prestressed building members. One is the all concrete structure utilizing precast or prestressed colmuns, prestressed beams, and prestressed channel or double-T roof sections. We believe channels will be used more and more in the future instead of double-T's because of better camber control and edge load transfer. Here the ideal and most competitive range is in bays from 30′ to 60′. In this range the all precast prestressed units fabricated at a plant and fastened together at the site will offer certain advantages.

In addition to the all concrete structure is the possible use of pretensioned beams and precast columns as the framework for a structure. This then permits the use of various other types of cover, such as sheathing, decking, etc., spanning between beams. Such a design would be in competition with the normal steel framework which is used so commonly. Fire resistance with prestressed concrete girders and wood sheathing would be better than with steel and wood sheathing.

We believe this latter application will be a definite trend in the future, largely because of the number of such structures as opposed to the limited number of all concrete buildings which are erected. Based on present experiences in various parts of the country, it appears that the prestressed concrete industry will be able to compete, economically and structurally, with such steel frame designs in many instances.

As I mentioned earlier, there will be unusual and novel applications of prestressed concrete in the building field in the future. For example, there is currently in the planning stage a 16 story building for Northern California which utilizes precast prestressed elements spanning 60 ft. Prestressed arches are definitely feasible. With proper design and technique, units can be prestressed on curves and angles as well as in straight lines. This applies both to pretensioned and posttensioned members.

While the unusual and novel have, in many fields of endeavor, become the commonplace, our crystal ball for the future shows the volume use of prestressed concrete building members to be, as indicated, in low level structures which permit plant manufacture of standard units of high quality, in volume, for quick and efficient erection at the site.

MARINE PROJECTS

Wherever forming over water can be eliminated by the use of precast concrete elements, a considerable savings in time and money can be achieved. While the use of prestressed concrete elements for marine projects is merely an extension of an old idea to a new material, there are many advantages in the use of prestressed concrete over regular precast units.

To begin with prestressed concrete is a crack free material which, of course, is extremely important in any structure over water. Prestressed concrete will permit wider bent spacings without the large increase in weight and size normal to longer spans in regular precast concrete. Prestressed units will be easier to handle because of their greater inherent strength and will provide a stronger structure.

We are now working on a small marine project which is completely prestressed – piles, caps, stringers and flat slab deck units. The only cast-in-place concrete will be for connections. Such structures will become increasingly common in the future with a trend to the largest units which can be economically handled by available equipment.

Today individual units for marine projects, such as caps, stringers, and deck units, are prefabricated at a plant and assembled at the site. In the near future such units will be assembled at a plant in large sections comprising perhaps 2, 3, or 4 bents, barged to the site, set on previously driven piles and necessary connections made. Here the possibility exists of setting large flat slabs directly onto piles thereby eliminating caps and stringers. This would be very similar to the lift slab technique for buildings. Wher-

ever water can be used as a means of transporting precast and prestressed units, the only limitation on size and shape will be the equipment available for transporting and handling the units.

Under marine projects we must also consider as a possible future field for prestressed concrete, off-shore drilling and oil storage platforms. Prestressed concrete has been successfully used on several such structures on the Gulf Coast. Its use on similar projects off California coasts is now under consideration.

Prestressed concrete development for marine projects, such as piers, wharves, platforms, etc., will, in all probability, parallel that of bridge members — larger units in standard sections. Standard designs for caps, connections and flat slab deck units are now under design study by our Engineering Department and will be published in the future.

PILES

The structural strength which is given to a member by prestressing is nowhere more apparent than in pretensioned concrete piles.

Based upon our experience and official tests, pretensioned concrete piles have been proven to be an entirely new type of pile — just as prestressed concrete is a new material.

While it has the outward appearance of a precast concrete pile, the handling, driving and strength characteristics of this pile are closer to those of a steel H-pile. Without question, it is the finest concrete pile available today and equal to steel H-piles under many conditions.

At Eureka, California, on a project for the State Division of Highways, we drove and pulled a 20″ square pretensioned concrete test pile five times with no apparent damage, driving to 100 tons indicated bearing each time.

At San Francisco, on a project for the State Harbor Board, we drove 140 20″ octagonal pretensioned piles, 132 ft. long, through 30 ft. of riprap with no apparent damage.

At Long Beach on a test project for the Port of Long Beach and the U. S. Naval Research and Evaluation Laboratory, a 14″ octagonal pretensioned pile was driven in 30 ft. of water intermittently to a final penetration of 398 blows to the inch with no damage whatsoever reported by the diver. An identical 14″ pile was driven continuously to 92 blows to the inch with no damage reported by the diver. In the same test 18″ square standard precast piles were driven to a maximum of 140 blows to the

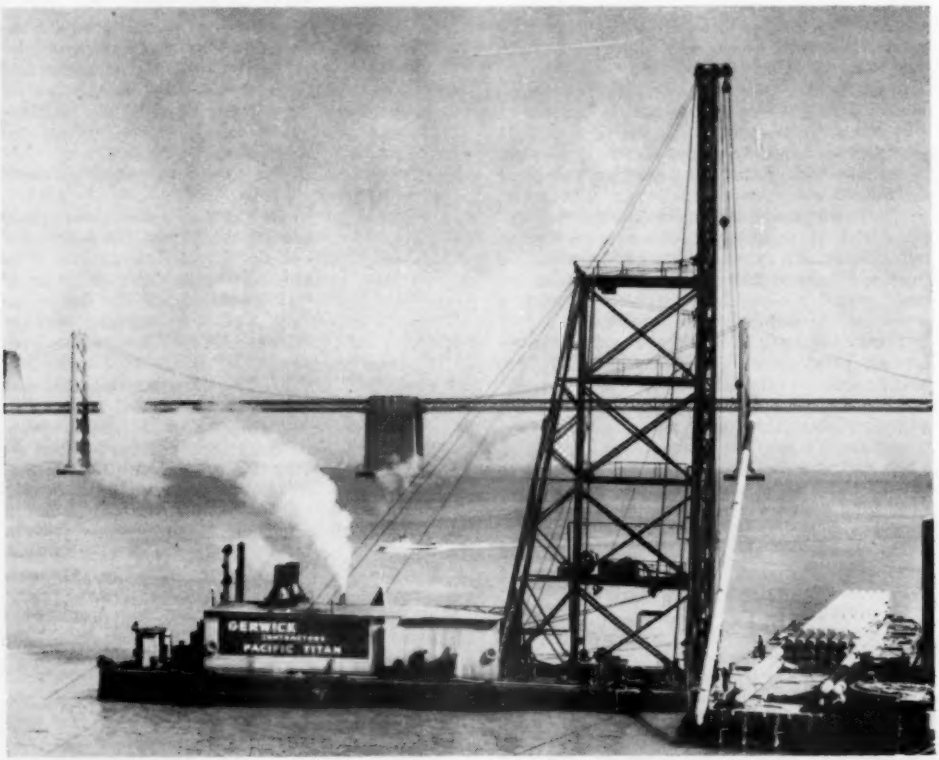


FIG. 48A. Lifting 132′ long 20″ octagonal pretensioned pile off barge, for Pier 15-17 project, San Francisco (April, 1956). Over 2,000 pretensioned piles used in this project, both 14″ octagonal and 20″ octagonal.

inch but with major structural damage.

At Richmond, California, 16″ octagonal pretensioned concrete piles are being used in lieu of 12″ BP 53# steel H-piles (with a 24″ concrete shell) to drive through a dense layer of sand gravel. Results are excellent — the savings in money was considerable.

These factual cases are rather conclusive. The question is, therefore, how can these facts be translated into usable data. These are the conclusions which we have reached, based upon use, study and analysis.

1. Pretensioned concrete piles have been, and can be, used in lieu of steel H-piles in many instances. Our study indicates a 14″ octagonal to be equivalent to a 12″ BP 53#, a 16″ octagonal equivalent to a 14″ BP 73#, and 20″ octagonal equals a 14″ BP 102#. For penetration into rock, a steel H stub has been placed in the tip of the pile in some projects, thereby gaining the structural advantages of both piles.

2. Where concrete piles are required in marine construction, there is no adequate substitute for a pretensioned pile. The crack-free, high bending strength and durability against erosion and corrosion of these piles cannot be equaled in any other type of pile.

3. Because of the great strength of pretensioned concrete piles, it is possible to reduce the section substantially as compared to regular precast piles and still have a pile which is structurally superior to a regular precast pile. This was conclusively shown at the Long Beach pile tests where a 14″ octagonal pretensioned pile proved to be far superior to an 18″ square precast pile. Of particular importance is that a 14″ pretensioned pile will compare favorably in initial cost with an 18″ precast pile and will be easier to handle and drive.

4. Pretensioned concrete piles can be efficiently and economically used for building foundations, as well as marine projects. For example, if soil conditions permit, a 12″ octagonal pretensioned pile can be used as a 54-ton pile, with safety factor of about 4 to 1. This pile is greater in volume, perimeter, point bearing area, concrete strength and reinforcing than various types of cast-in-place concrete piles with a comparable rated bearing value. Its quality is a known factor before it ever leaves the plant. Wherever they can be efficiently used on land projects the cost per ton carried will be quite low.

As with any pile, however, there are certain places where pretensioned piles are not feasible for use. For very short piles and where cut-offs are highly irregular, these piles are not as economical as other types of piles.

Future trends in the prestressed concrete pile field indicate that larger piles and greater bearing capacities for smaller, as well as larger piles, will be increasingly used wherever possible. As you may know, 54″ diameter prestressed piles were used on the Lake Pontchartrain Bridge with successful results. Other projects are under consideration for 36″ and possibly larger piles. Wider bent, or footing spacings and greater batters will be used as a result of the greater structural strength of this pile.

The use of pretensioned concrete sheet piles is also an excellent application which has been recently developed. In marine work their use as permanent bulkheads will prove economical and sound.

Another possible application of pretensioned prestressed concrete which we have studied is that of a fender pile. They have practically all of the properties which a fender pile must have.

A prestressed concrete fender pile would be trapezoidal in shape, say 12″ on top, 10″ on the bottom and 12″ deep, or similar dimensions. This would permit double bolting to resist torsion. The ability to resist longitudinal and lateral shear would be excellent. The skin friction surface available would be a maximum. Such a fender pile would have good resistance to abrasion, be impervious to marine borers, be easy to handle, drive and bolt. They would be absolutely uniform and strong in bending.

Economic studies indicate that they would definitely be competitive with creosoted timber piles because of their much greater strength and longer life.

For friction piles various shapes of pretensioned concrete piles are possible which will give a higher frictional area to volume ratio than can be obtained with the usual square, octagonal or round piles. Pretensioned friction piles in the shape of T's and Y's have been successfully used in the Far East. We have recently designed four and five pointed "star piles" which have excellent ratios of weight and volume to skin friction area, and which can be produced, handled and driven very economically.

There has been tremendous interest in pretensioned concrete piles here on the West Coast as well as elsewhere. Because they meet the structural and economic criteria previously mentioned, they are included in numerous projects now on the boards. The future use of these piles appears almost unlimited.

MISCELLANEOUS USES

These applications of prestressed concrete which I have just discussed are common and well known to most of you. Piles, bridges, buildings and marine projects are all structures which lend themselves to prefabrication.

There are other uses of prestressed concrete which have great possibilities for the future. In fact, there is hardly a place where concrete is used that prestressed concrete cannot be used. And, there are some places where cast-in-place concrete is not feasible but which will be ideally suited for prestressed concrete.

In addition to this competitive situation with reinforced concrete, prestressed concrete will often be used in place of steel or timber on many projects because of structural and economic reasons. Let me point out a few of these "miscellaneous" applications which I believe will develop in the future.

1. *Prestressed Concrete Barges:* Concrete has been used for barges before but the use of prestressed concrete will make their construction much more feasible and economical. We know they will be competitive with steel. In fact, we have made preliminary designs for a very large self-propelled freight vessel in prestressed concrete for use in the Bay Area which indicates the practicability of such construction. The manufacture of prestressed concrete barges, towed and self-propelled, would be an industry in itself.

2. *Overhead Transmission Lines:* All technicians concerned with the transfer and distribution of electric power have given and still do give attention to the important problem of "conductor supports" for transmission lines. It is essential to obtain reliable and long-lasting transmission of power with the least possible depreciation. Let me quote from an article in a recent French publication by Mr. P. Lombard, Chief Engineer of the Gas and Electricity Development Service in Algeria. After a brief discussion of wooden poles, flange metal girders, metal lattice pylons and ordinary reinforced concrete posts for transmission lines, he drew the following conclusions: "Inspection of the first (prestressed posts manufactured and erected in 1941 has incidentally revealed that they have held up perfectly with the passage of time."

"From the technical point of view, the prestressed posts are considerably superior to ordinary concrete posts since they develop no cracks, even of a capillary nature, and the reinforcement is therefore protected from any penetration of water. It is impossible for such posts to be destroyed through oxidation of the rods, and their life is therefore practically unlimited. Besides, they are lighter, less cumbersome, stand up better under transportation and handling, and finally, are more elegant in appearance."

"On the economic side, it has been shown above that they were less expensive than of either ordinary reinforced concrete, wide metal girders or metal pylons."

"Furthermore, they require no maintenance. They are therefore not only more economical in their initial cost but also cheaper to maintain." I believe the above excerpts need no further comment other than to point out that all prestressed concrete products generally have such advantages.

3. *Landing Runways and Aprons:* In the immediate future I believe we will see a trend to construction of runways of prestressed concrete. Both the Navy and Army have test strips of prestressed concrete in operation today. You may have read of prestressed runways in France and Algiers which were built of unreinforced concrete, longitudinally compressed with flat jacks and transversely post-tensioned. In this country, I believe the greater ultimate strengths obtained with strands will lead to the use of longitudinal pretensioning with transverse post-tensioning. Abutments will be constructed at intervals from 1000 to 5000 ft., strands will be pulled and stressed, and the concrete slab poured. Such methods will result in structurally stronger runways with a considerable savings in concrete over the 18"-24" slabs which are being used today.

In view of the increasing size of today's aircraft and the loads which the runways must carry, this will be an exceedingly important development. It is quite conceivable, also, that this development will extend to concrete highways, as it has in England.

These are but a few of the fringe area developments today in prestressed concrete which will become common practice tomorrow. If you, as engineers, have ideas on other applications, any leading manufacturer will be glad to furnish you information and data.

CONNECTIONS AND CONTINUITY

Connections which tie prefabricated units together at the site are as important a part of the design as the units themselves. While such connections are a vital part of any structure, they become doubly important here in the West where seismic conditions exist.

Generally speaking, connections for prestressed units fall into four categories: (1) welding, (2) bolting, (3) mild steel with cast-in-place concrete and (4) post-tensioning. Each type of connection will have application in different types of structures both for simple spans and continuity development. The right connection will be dependent on the type of member and the conditions involved.

The greatest progress in the matter of connections remains to be made in the field of continuity. In order to make full use of the material continuity should be developed, where possible, to achieve maximum efficiency. While various methods of developing continuity have been used for prestressed concrete units it is still a field which demands further study and research.

Past experience has shown that simple spans of prestressed concrete can be competitive with other materials used as continuous members. This competitive position will improve as further advances are made in development of continuity of prestressed concrete units.

TRANSPORTATION AND ERECTION

Hauling of precast prestressed concrete has different limitations depending upon the means of transportation. Truck transportation limits the length of members which can be transported to about 80 ft. Members up to 150 ft. could be transported by rail whereas practically no limitation exists on water transportation.

The distance which it is possible to ship economically is difficult to specify since it will vary from product to product, different destinations and depend on competition which may be present at that point.

We have quoted competitively on piles for shipment to midwest, south and to Alaska. On bridge and building members as well as piles, we have quoted competitively on proj-

ects throughout California.

I have been informed that prestressed concrete building units will soon be shipped from Napa to Seattle for a large project covering close to 500,000 square feet. This project probably seems large enough to set up for at the site. However, shipping costs to distant points will often be offset by the more economical and efficient operation and productivity which can be attained in a central plant. This proved to be the case in this project. The ability of the prestressed concrete industry to economically ship considerable distances should not, therefore, be underestimated.

Many contractors penalize prestressed concrete erection quotations because of unfamiliarity with it. While this is natural with any new material, it is expected that such "penalties" will diminish as experience is gained in handling such units. This, of course, can only come with time. Whenever that experience has been gained, erection has become a simple problem, as indeed it is. For example, on three recent jobs on which we furnished prestressed concrete units the contractors underestimated their ability to erect by scheduling shipments over twice as long a period as was necessary.

CONCLUSION

Before closing I would like to clarify a few points which may be possible of misinterpretation.

My comments to you today have been made, not only as a fellow member of the Structural Engineers Association, but as a manufacturer of prestressed concrete and as an officer of the Prestressed Concrete Institute.

I have attempted to briefly describe what the industry has done, what it is doing in the development of manufacturing techniques, and some of the trends and possibilities which we, as manufacturers, see in the future.

While we may plan, predict and hope about the future of prestressed concrete, we recognize that it will be up to the design engineer to give form to these ideas. Therefore, our goal will be to make it as easy as possible for you to consider, accept, and use prestressed concrete in your projects whenever structurally or economically feasible.

The prestressed concrete industry is naturally quite enthusiastic about its products. Properly used they will be the ideal solution to many of your problems.

The industry recognizes that its products must be economically competitive and structurally sound to be acceptable. However, we ask that you recognize this fact – prestressed concrete is a high quality product produced under close supervision of materials and workmanship. As such it may well be worth the additional cost, as a better product, in those instances where another material is apparently more economical.

We hope that you do not feel we are infringing upon your design integrity when we propose various standard designs. Our purpose is two-fold in doing so – to achieve an economy in manufacturing through standardization and to save you design time. We will, of course, make any type of prestressed concrete product of your own design if our standard sections are not adaptable to your project.

At the present, standard design products vary to some extent from company to company. It is hoped that in the near future– it may be one year or five years – the Prestressed Concrete Institute, as the voice of the industry, will be able to publish a national standard handbook on different members, codes and specifications. Until that time any manufacturer of prestressed concrete will be glad to discuss with you possible prestressed concrete projects without obligation.

It is a pleasure to tell you that next July 29th through August 3rd the Prestressed Concrete Institute will hold its 3rd annual meeting in San Francisco in conjunction with a World Conference on Prestressed Concrete sponsored by the University of California. We sincerely hope you will be able to attend as the program will, I am sure, be most interesting to you.

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By J. Rex Farrior, Jr.
Legal Counselor for P. C. I.
Tampa, Florida

P.C.I. is legally known as Prestressed Concrete Institute, Inc., a corporation not for profit organized and existing under the statutes of Florida, Chapter 617. The Institute is legally recognized as a scientific, charitable and benevolent association.

The original charter was approved by the Circuit Court at Tampa, Florida, on June 18, 1954. Officially, the object of the Institute is the formation of an association of manufacturers of prestressed concrete and of persons and companies in allied or related industries for the mutual exchange of information and ideas; and generally to do anything beneficial for its members and the prestressing business which does not violate the laws of Florida or the United States.

The charter provides for seven classes of members: Active, for prestressed manufacturers; Associate, for related businesses; Professional, for architects and engineers; Affiliate, for supervisory and technical employees, non-graduate; Junior, for architects and engineers in training; Student, for students enrolled in accredited architectural and engineering colleges, and Honorary, for such persons as the Board of Directors may wish to honor.

From a tax standpoint, the Institute enjoys an enviable position. Being a valid non-profit corporation, it is not required to pay any tax on its receipts, and therefore 100 per cent of its income can be devoted to research and other useful projects.

Fortunately, also each member's dues and contributions are completely deductible by the member for income tax purposes.

Both tax factors therefore guarantee to the Institute's members more than full value for each dollar invested.

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Anyone engaged in the production of prestressed concrete, or in the production of materials or equipment allied to the prestressing industry; licensed architects and engineers; engineers and architects in training, and students enrolled in accredited schools of architecture and engineering, are invited to apply for membership in the Prestressed Concrete Institute. Membership classifications have been established as follows:

ACTIVE
Organizations and individuals actively engaged in the production of prestressed concrete products.
Annual Membership Dues. **$250.00**

ASSOCIATE
Organizations and individuals engaged in the production of materials and equipment allied to the prestressed concrete industry.
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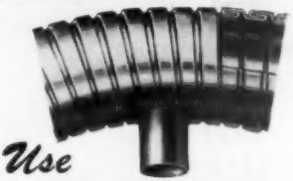

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The building illustrated is one of several in the Los Angeles area designed by architectural associates Jack H. McDonald and Cejay Parsons (one of Rockwin's many customers) utilizing economical prestressed double T concrete slabs for floors, ceilings and roofs.

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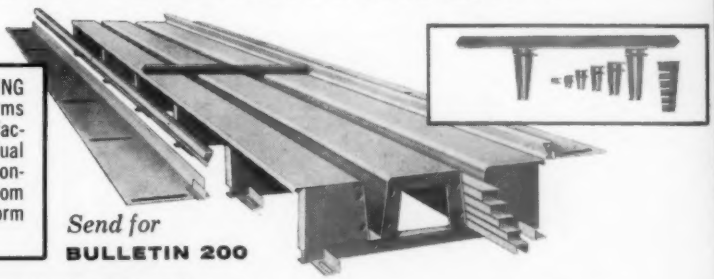

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